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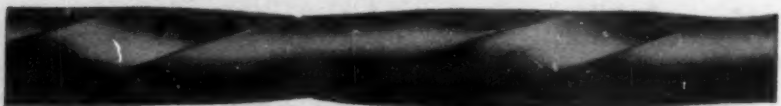

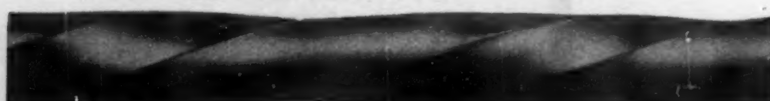

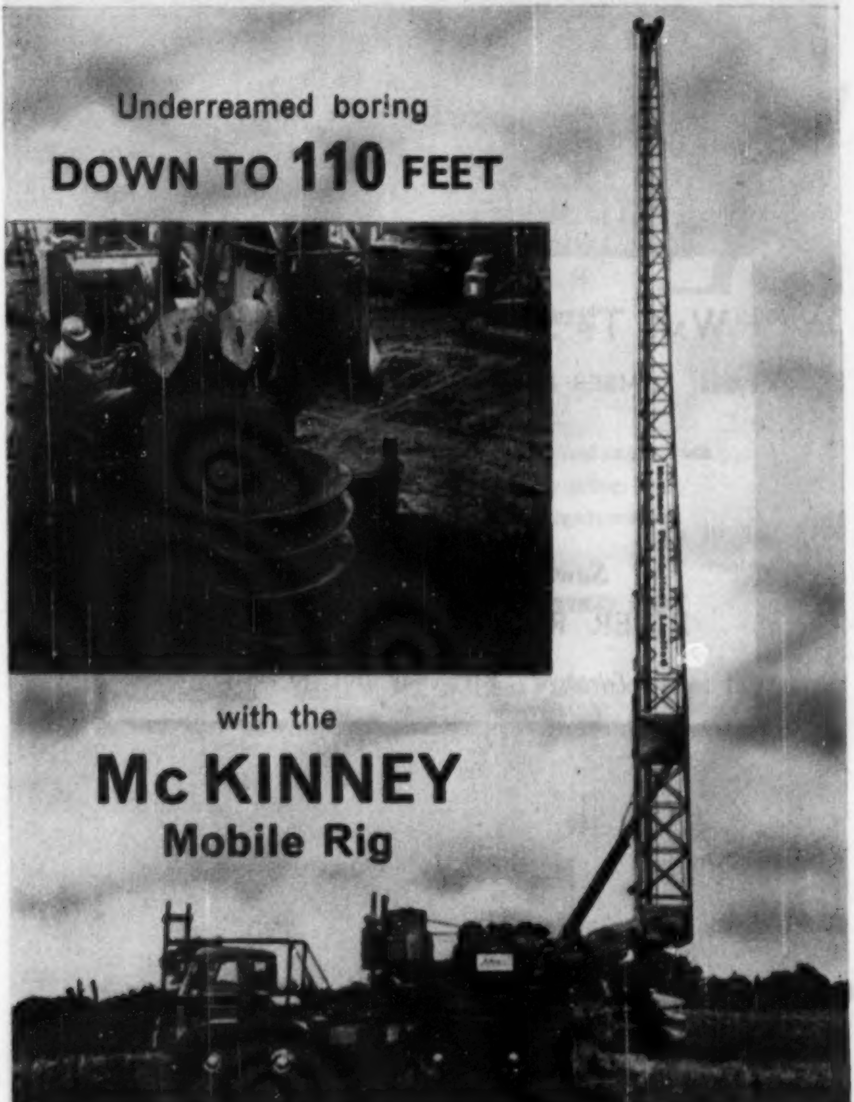
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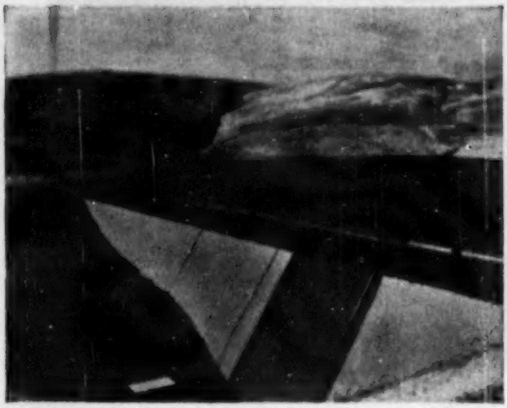

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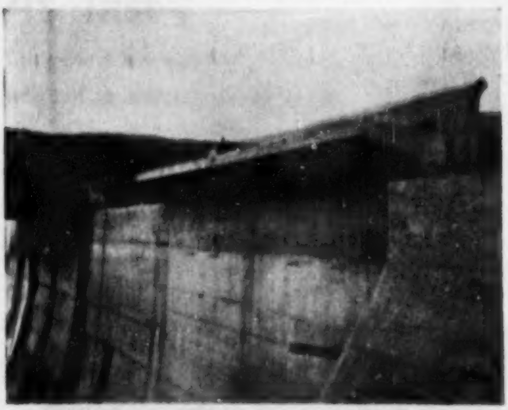

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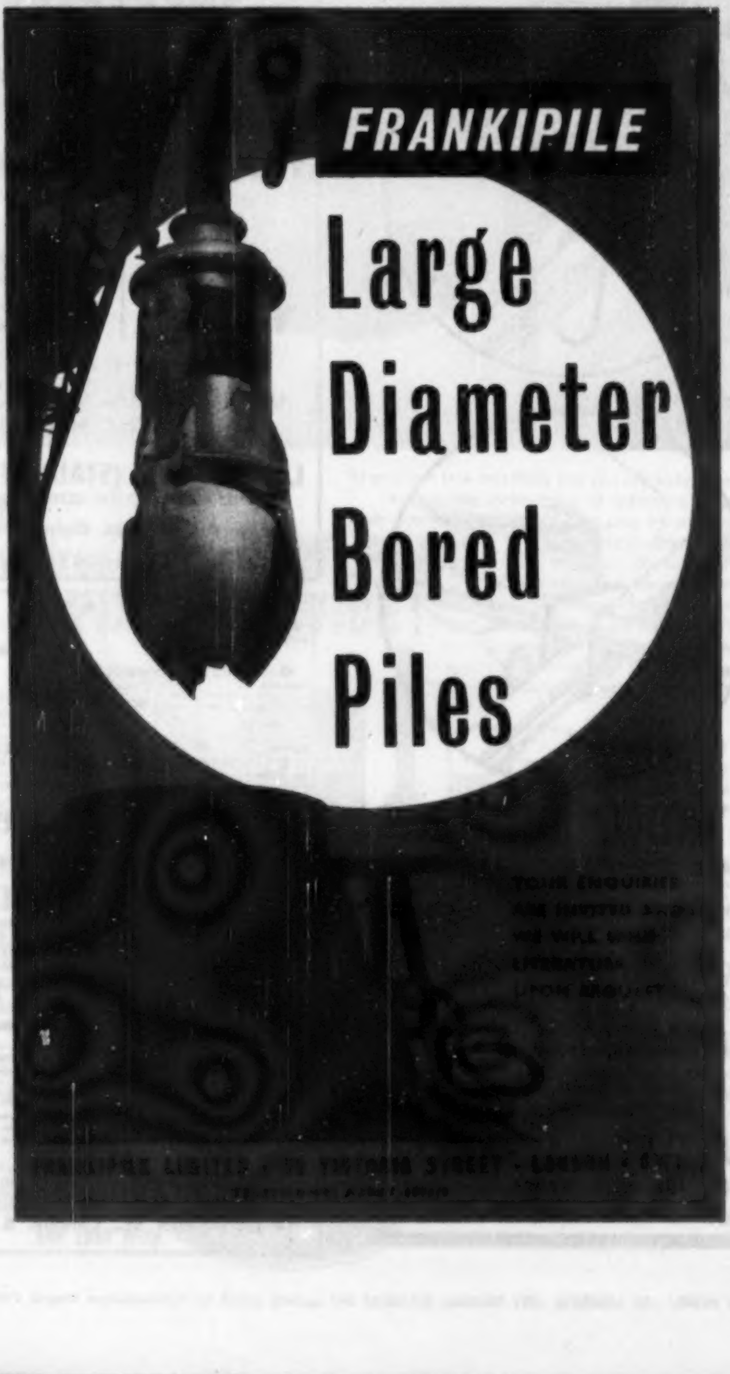
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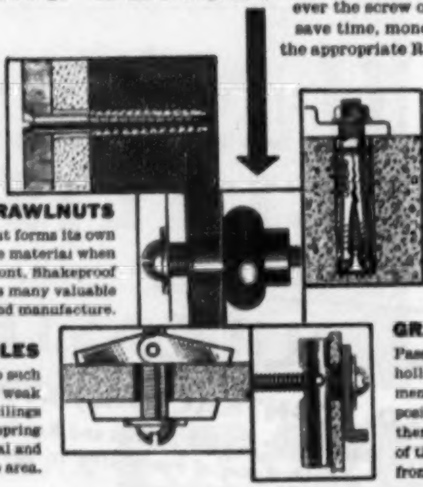

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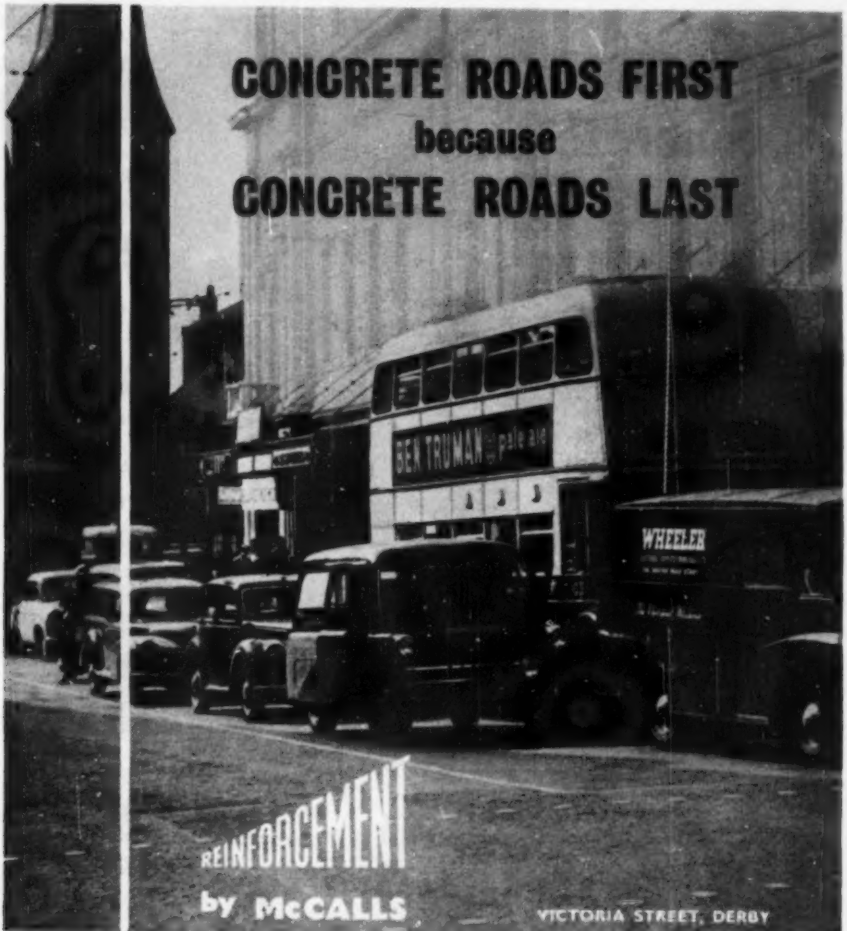
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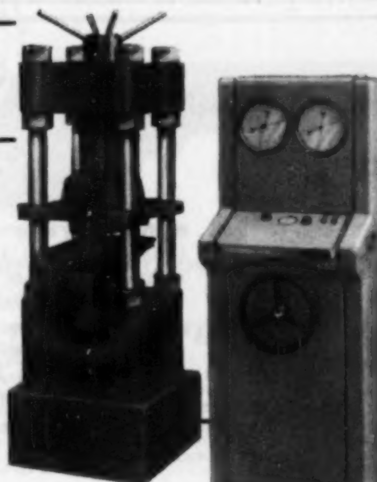

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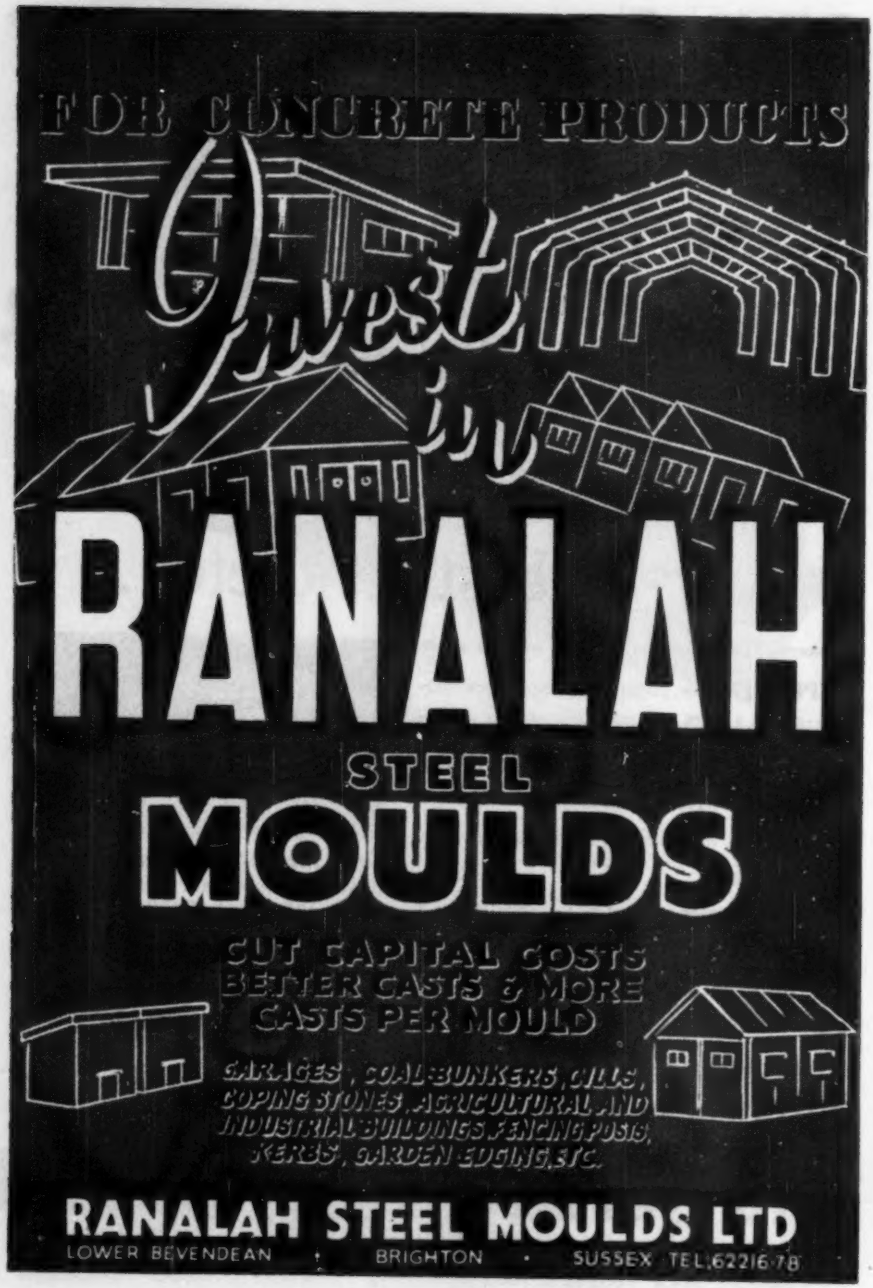
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CONCRETE
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EDITORIAL NOTES

Concrete in the Gas Industry.

The growing importance of civil engineering in the gas industry was emphasised in some of the papers presented to the International Gas Conference held in Rome recently. A representative of one of the British Gas Boards indicated the extent to which recent developments in civil engineering, and in concrete in particular, are used in structures associated with the production, storage, and distribution of gas, and also described how the British gas industry is applying knowledge gained in other industries and how these developments have been applied in ways that sometimes are new, such as site investigations, precast and prestressed concrete, and the foundations of vibrating machinery. In foundation construction, methods have been developed whereby work such as sheet-piling, which is generally a temporary provision, is incorporated as part of the permanent works. New methods of testing piles have also been investigated.

The cost of the foundations and other civil engineering works at the older types of gas-works may have been as much as 25 per cent. of the total cost, but in some of the more modern methods of making gas the cost of the civil engineering work is a smaller proportion of the total. A major requirement in civil engineering works at gas-works is resistance to corrosion by the atmosphere and the ground. For this reason one of the earliest applications of reinforced concrete was to replace steel for many such works. Although steel plates are still commonly used for gas-holders, it is interesting to note that fifty years ago a non-telescopic gas-holder in Germany was constructed completely of reinforced concrete and was described in this journal at the time. The first gas-holder tank of reinforced concrete in Great Britain was built about 1902 at Hampton Court.

Many gas-works are on sites where the ground has little strength and therefore the cost of foundations is an important proportion of the total cost of the civil engineering works. Because steel structures are lighter in weight the foundations for such structures are less costly, but the greater cost of maintenance of steel has resulted in the use of reinforced concrete superstructures where practicable, in spite of their greater weight. The virulent conditions that exist at gas-works, however, have sometimes resulted in the spalling and subsequent deterioration of reinforced concrete a few years after its construction. The advent of prestressed concrete was therefore welcomed by the gas industry because it enables a structure to be as much as 25 per cent. lighter in weight than one built of reinforced concrete,

and, if it is well designed, it is free from cracks which are commonly the first cause of deterioration of reinforced concrete. Even if prestressed concrete were more expensive than reinforced concrete, and this is not always the case—especially if the lower cost of the foundations is taken into account—the gas industry is likely to use prestressed concrete because of its greater durability in polluted atmospheres, and many prestressed structures have been and are being built at gas-works. It has been claimed that some of these structures or their components recently built in Great Britain are the first of their type: for example, it is thought that the first prestressed piles in Great Britain were driven in connection with a gas-holder tank at Stanford-le-Hope as reported in this journal for May, 1950. Prestressed concrete is also particularly suitable for the ring-beams of gas-holder tanks. In the past the crown-framing in such tanks was built in steel, but reinforced concrete, and more recently prestressed concrete, have been used for this purpose. There is already sufficient experience to show that fully-prestressed concrete is more resistant to corrosion than reinforced concrete, and the experience on railways suggests that the same may be said of concrete members that are partially prestressed. Thin concrete slabs as are usual in "shell" roofs are not particularly suitable at gas-works because of the thin cover of concrete over the steel. Thin slabs have, however, been used for the roofs of buildings at gas-works and chemical works but protection is necessary to ensure durability.

New developments such as the importation of methane, the gasification of certain products from oil refineries, German processes now in use in Great Britain for the complete gasification of coal, and a new British system of hydrogenation, may give the gas industry a new lease of life, particularly as the cost of distributing gas is very much less than the cost of distributing equivalent amounts of energy in the form of electricity. Some of the new methods of making gas require civil engineering works of types not formerly used at gas-works, although in general they may require less civil engineering work. Small bases only are required for the main plant used in the new processes, but water towers and chimneys may be required for auxiliary processes and the importation of methane would require jetties or wharves and large insulated storage tanks. Civil engineering activities are also likely to result from recent developments such as the investigation of the practicability of underground storage and proposals to centralise production.

In the case of the hydrogenation and other processes incorporating an oxygenation plant, extremely low temperatures are experienced. There is little data available regarding the behaviour of concrete at low temperatures. Research at the University of Ghent has shown that at a temperature of minus 40 deg. C. the crushing strength and modulus of rupture of concrete increases considerably, as does the ultimate strength of a beam, and that, consequent upon the increase in the elastic modulus, there is less deformation. At this temperature mild steel becomes brittle but, even at temperatures as low as that of liquid nitrogen (about minus 200 deg. C.), data from U.S.A. sources show that high-tensile steel seems to suffer little loss of strength or ductility. The temperature in some of these new plants may be below minus 150 deg. C., and in the present state of knowledge it may be advisable to insulate the concrete so that it is subjected only to temperatures at which its behaviour is predictable. Such insulation would also reduce the stresses due to differences of temperature, and avoid the possibility of unknown phenomena which may occur at very low temperature.

The Analysis of Symmetrical Single-bay Multiple-story Frames.

By K. BRZEZINSKI.

For any arrangement of loads, a single-bay multiple-story frame which is symmetrical can be analysed by the moment-distribution method using two operations only, namely one for the symmetrical component of the load and the other for the asymmetrical component. It is necessary to calculate the moments for only half the frame, usually the part to the left of the axis of symmetry.

Symmetrical Loads.

For symmetrical loads the stiffness of any beam is $K_b = \frac{2EI}{L}$, the stiffness of a column with both ends fixed is $K_c = \frac{4EI}{L}$, the stiffness of a column with one end hinged is $K_c = \frac{3EI}{L}$, and the carry-over factor is $C = +\frac{1}{2}$.

The fixed-end moments for most common arrangements of loads on beams are given in text books and it is assumed that the method of moment distribution is known. The sign conventions for external loads are that vertical loads are positive when acting downwards, that horizontal loads are positive when acting from left to right, and that moments are positive when acting clockwise. The moments for one half of the frame will be of opposite sign to the moments for the other half.

Asymmetrical Loads.

(1) Frames with Vertical Columns Only.—The stiffness of a beam is $K_b = \frac{6EI_b}{L_b}$, the stiffness (defined as the moment required at the free end of the column to rotate it through unit angle, while the other end remains fixed) for each end of a column with both ends fixed is $K_c = \frac{EI_c}{L_c}$, but for the top of a column which is hinged at the bottom $K_c = 0$. The carry-over factor for columns is $C_c = -1$; there is no carry-over factor for beams or for the hinged end of a column.

The fixed-end moments for the asymmetrical component of the load on a beam are given in tables, or can be calculated directly from the fixed-end moments M_A and M_B produced on a beam AB by the total load.

For symmetrical loads, $M_A' = \frac{M_A + M_B}{2}$.

For asymmetrical loads, $M_A'' = \frac{M_A - M_B}{2}$.

The fixed-end moments for the columns are calculated on the assumption that the top of the column is fixed against rotation only and is free to move horizontally. The column acts as a cantilever, and the moments for common

TABLE I.—LOAD FACTORS FOR CANTILEVERED BEAMS.

1	2	3	4	5	6
A B $M_A = M_A^\circ - M_B$	F.E. Moment M_A°	Type of loading	Rotation θ_B	Deflection y_B	A B $M_B = -\theta_B \cdot \frac{EI}{L}$
0	$-M$	A L B M	$+\frac{ML}{EI}$	$+\frac{ML^2}{2EI}$	$-M$
$-M \cdot \frac{b}{L}$	$-M$	A a M b B	$+\frac{Ma}{EI}$	$+\frac{Ma^2}{2EI}\left(1+\frac{2b}{a}\right)$	$-M \cdot \frac{a}{L}$
$-\frac{PL}{2}$	$-PL$	A P	$+\frac{PL^2}{2EI}$	$+\frac{PL^3}{3EI}$	$-\frac{PL}{2}$
$-Pa\left(1-\frac{a}{2L}\right)$	$-Pa$	A P a b B	$+\frac{Pa^2}{2EI}$	$+\frac{Pa^3}{3EI}\left(1+\frac{3b}{2a}\right)$	$-\frac{Pa^2}{2L}$
$-\frac{WL}{3}$	$-\frac{WL}{2}$	A W B	$+\frac{WL^2}{6EI}$	$+\frac{WL^3}{8EI}$	$-\frac{WL}{6}$
$-\frac{Wa}{2}\left(1-\frac{a}{3L}\right)$	$-\frac{Wa}{2}$	A W a b B	$+\frac{Wa^2}{6EI}$	$+\frac{Wa^3}{8EI}\left(1+\frac{4b}{3a}\right)$	$-\frac{Wa^2}{6L}$
$-\frac{WL}{4}$	$-\frac{WL}{3}$	A W L B	$+\frac{WL^2}{12EI}$	$+\frac{WL^3}{15EI}$	$-\frac{WL}{12}$
$-\frac{Wa}{3}\left(1-\frac{a}{4L}\right)$	$-\frac{Wa}{3}$	A W a b B	$+\frac{Wa^2}{12EI}$	$+\frac{Wa^3}{15EI}\left(1+\frac{5b}{4a}\right)$	$-\frac{Wa^2}{12L}$
$-\frac{5}{12}WL$	$-\frac{2WL}{3}$	A W L B	$+\frac{WL^2}{4EI}$	$-\frac{11}{60}\frac{WL^3}{EI}$	$-\frac{WL}{4}$
$-\frac{2Wa}{3}\left(1-\frac{3a}{8L}\right)$	$-\frac{2Wa}{3}$	A W a b B	$+\frac{Wa^2}{4EI}$	$+\frac{11Wa^3}{60EI}\left(1+\frac{15b}{11a}\right)$	$-\frac{Wa^2}{4L}$

cases can be found from *Table* I. If the column is hinged at the bottom the moment at the top is M_A°.

In addition to the fixed-end moments caused by the direct horizontal loads on the column, there is a sway moment M_s caused by all the horizontal forces ΣP acting at, or above, the top. At each end of a column with both ends fixed, $M_s = -\frac{L}{2} \times \Sigma P$; at the top, with a hinged bottom, $M_s = -L \times \Sigma P$.

For asymmetrical loading the restraint moments have the same sign for both halves of the frame.

(2) FRAMES WITH INCLINED COLUMNS.—The stiffness of a beam is, as previously, $K_b = \frac{6EI_b}{L_b}$. The stiffnesses of a column fixed at both ends are :

$$\text{At top, } K_h = \frac{3EI_c}{L_c} \times \frac{m^2}{1 + m + m^2}$$

$$\text{At bottom, } K_f = \frac{3EI_c}{L_c} \times \frac{1}{1 + m + m^2}$$

in which m is the ratio of the distance between the tops of the columns to the distance between the bottoms. For columns hinged at one end, as before $K_h = K_f = 0$.

The carry-over factors for columns with both ends fixed are : From top to

bottom, $C_{hf} = -\frac{1}{m}$. From bottom to top, $C_{fh} = -m$. There is no carry-over factor for beams or for columns with one end hinged.

Moments due to Sway.

Top Story.—Considering the top story (*Fig.* 1) and assuming that the bottom of the column A_2 is hinged, the shearing force S_1 at the bottom is found by balancing the moments of all the forces acting on the top part of the frame about O.

$$S_1 \times (L_0 + L_1) + P_1 \times L_0 \times \sin\alpha + N_1 \times L_0 \times \cos\alpha = 0 \quad . \quad . \quad (1)$$

from which $S_1 = -(P_1 \times \sin\alpha + N_1 \times \cos\alpha) \times m_1$.

The sway moment for the column, if it is hinged at the bottom, would be

$$M_1^0 = S_1 \times L_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

but if both ends are fixed, and the stiffnesses and carry-over factors are known, it can be shown that the sway moment at the top is

$$M_h = M^0 \times \frac{2 + m}{2(1 + m + m^2)} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and at the bottom

$$M_f = M^0 \times \frac{1 + 2m}{2(1 + m + m^2)} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Allowance should be made for any asymmetrical load on beam A_1B_1 in equation (1) to obtain the total shearing force.

$$\sin\alpha = \frac{h_0}{L_0} = \frac{h_1}{L_1}$$

$$\cos\alpha = \frac{b_1}{2L_0}$$

$$m_1 = \frac{b_1}{b_2} = \frac{L_0}{L_0 + L_1} = \frac{h_0}{h_0 + h_1}$$

$$m_2 = \frac{b_2}{b_3} = \frac{L_0 + L_1}{L_0 + L_1 + L_2}$$

$$m_1 \times m_2 = \frac{L_0}{L_0 + L_1 + L_2}$$

Fig. 1.

Lower Stories.—As for the top story, the frame is assumed to be cut at A_3 and the moments of all the forces on this part are balanced about O.

$$S_2 \times (L_0 + L_1 + L_2) + P_1 \times L_0 \times \sin\alpha + N_1 \times L_0 \times \cos\alpha + P_2 \times (L_0 + L_1) \sin\alpha + N_2 \times (L_0 + L_1) \times \cos\alpha = 0$$

from which $S_2 = S_1 \times m_2 - (P_2 \times \sin\alpha + N_2 \times \cos\alpha) \times m_2$. For the next story below, $S_3 = S_2 \times m_3 - (P_3 \times \sin\alpha + N_3 \times \cos\alpha) \times m_3$. The fixed-end moments due to sway are obtained from formulæ (2), (3), and (4).

Fixed-end Moments due to a Transverse Load on a Column.

The fixed-end moments can be computed by finding the angular deformation at the bottom of the column under load, assuming that the column forms part of a simply-supported beam with one support at the bottom of the column and the other at the point of intersection O (the part from the head to O is considered to be infinitely stiff). The calculation is lengthy and the resulting formulæ are complicated, but an asymmetrical load does not often occur except at joints.

For a uniformly-distributed load W (on the full length) the fixed-end moment for a column hinged at the foot is $M_h^0 = -\frac{WL}{2} \times m$.

For a column with both ends fixed, the fixed-end moments are:

$$\text{At bottom, } M_f = -\frac{WL}{8} \times \frac{1 + 3m + 4m^2}{1 + m + m^2}.$$

$$\text{At top, } M_h = -\frac{WL}{8} \times \frac{3m + m^2}{1 + m + m^2}.$$

If a force P acts at right-angles to a column at a distance aL from the bottom, the fixed-end moments are:

For a column hinged at the foot, $M_h^0 = -PaL \times m$. For a column with both ends fixed:

$$\text{At the bottom, } M_f = -PaL \times \frac{(2 - 3a + a^2) + (2 - a^2)m + 2m^2}{2(1 + m + m^2)}.$$

$$\text{At the top, } M_h = M^0 - M_f \times m,$$

which is the general formula for the moment at the top of a column if the moment at the bottom is known.

For a concentrated load (force or moment) it is sometimes more convenient to introduce an imaginary beam of zero stiffness at the point of application of the load instead of trying to derive complicated formulæ, and to calculate the shearing force and moments and proceed with further calculations as for an actual beam.

Practical Application.

The example which follows explains the procedure and shows the calculations.

When there are several frames of which the top portions are identical and which differ only in height, for example bridge piers or trestles for an inclined conveyor, the work can be reduced by numbering the stories from top to bottom and making the calculations in the same order so that the calculations for the bottom part only of each frame will be different. This method can also be used

to analyse frames in which some columns are vertical and others inclined, but care must be taken to use the correct value of the shearing force where change of inclination occurs. This also applies when the top story forms a gable. All the formulæ in this article apply to prismatic members only.

A coefficient can be applied to the stiffness of the beam to allow for the effect of haunching. An abrupt change of the cross section of a column can be dealt with assuming that an imaginary beam of zero stiffness exists at the point of change. For gradual changes of section it is difficult to derive suitable formulæ; the designer must decide for himself the simplifying assumptions that can be made.

Example.

The dimensions and arrangement of loads are shown in *Fig. 2.* All members have the same section, and it is assumed that $EI = 1$;

$$\sin\alpha = \frac{12}{12 \cdot 5} = 0 \cdot 96\,; \quad \cos\alpha = \frac{3 \cdot 5}{12 \cdot 5} = 0 \cdot 28.$$

The stiffnesses of the beam $= K_b = \dfrac{6EI_b}{b}$; therefore $K_1 = \dfrac{6 \times 1}{6} = 1$,

$$K_2 = \frac{6 \times 1}{13} = 0 \cdot 46, \text{ and } K_3 = \frac{6 \times 1}{20} = 0 \cdot 3.$$

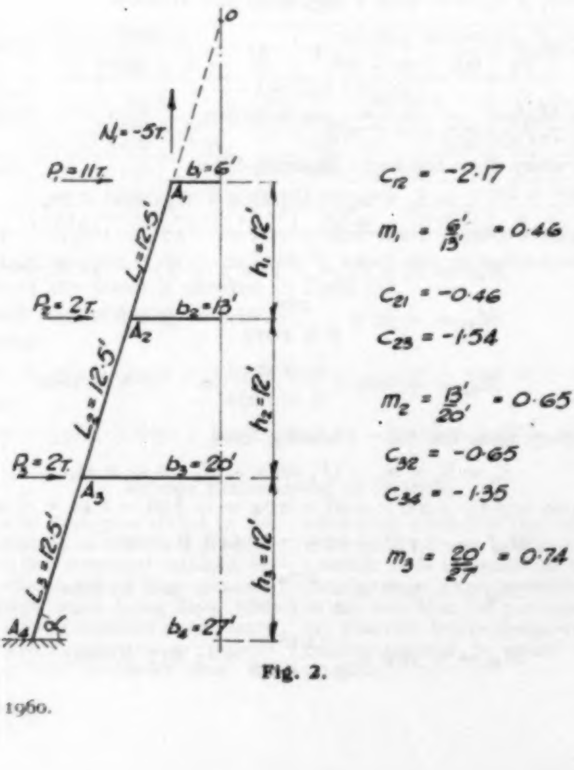


Fig. 2.

The stiffnesses of the columns are $K_h = \frac{3EI_c}{L_c} \times \frac{m^2}{1 + m + m^2}$

and $K_f = \frac{3EI_c}{L_c} \times \frac{1}{1 + m + m^2}$.

First story from the top:

$$K_{12} = \frac{3 \times 1}{12 \cdot 5} \times \frac{0 \cdot 46^2}{1 + 0 \cdot 46 + 0 \cdot 46^2} = 0 \cdot 24 \times \frac{0 \cdot 212}{1 \cdot 672} = 0 \cdot 030. \quad K_{21} = \frac{0 \cdot 24}{1 \cdot 672} = 0 \cdot 144.$$

Second story from the top:

$$K_{23} = 0 \cdot 24 \times \frac{0 \cdot 65^2}{1 + 0 \cdot 65 + 0 \cdot 65^2} = 0 \cdot 24 \times \frac{0 \cdot 422}{2 \cdot 072} = 0 \cdot 049. \quad K_{32} = \frac{0 \cdot 24}{2 \cdot 072} = 0 \cdot 116.$$

Third story from the top:

$$K_{34} = 0 \cdot 24 \times \frac{0 \cdot 74^2}{1 + 0 \cdot 74 + 0 \cdot 74^2} = 0 \cdot 24 \times \frac{0 \cdot 548}{2 \cdot 288} = 0 \cdot 058.$$

FIXED-END MOMENTS.—

First story from the top. Shearing force $S_1 = -(P_1 \sin \alpha + N_1 \cos \alpha) \times m_1$.

$$S_1 = -(11 \times 0 \cdot 96 - 5 \times 0 \cdot 28) \times 0 \cdot 46 = -(10 \cdot 56 - 1 \cdot 40) \times 0 \cdot 46 = -4 \cdot 22 \text{ tons.}$$

$$M_{12}^0 = S_1 \times L_1 = -4 \cdot 22 \times 12 \cdot 5 = -52 \cdot 8 \text{ ft.-tons.}$$

$$M_{12} = M_{12}^0 \times \frac{2 + m}{2(1 + m + m^2)} = -52 \cdot 8 \times \frac{2 \cdot 46}{2 \times 1 \cdot 672} = -38 \cdot 8 \text{ ft.-tons.}$$

$$M_{21} = M_{12}^0 \times \frac{1 + 2m}{2(1 + m + m^2)} = -52 \cdot 8 \times \frac{1 \cdot 92}{2 \times 1 \cdot 672} = -30 \cdot 3 \text{ ft.-tons.}$$

Second story from the top. Shearing force

$$S_2 = S_1 \times m_2 - (P_2 \sin \alpha + N_2 \cos \alpha) \times m_2.$$

$$S_2 = -4 \cdot 22 \times 0 \cdot 65 - 2 \cdot 0 \times 0 \cdot 96 \times 0 \cdot 65 = -2 \cdot 75 - 1 \cdot 25 = -4 \cdot 00 \text{ tons.}$$

$$M_{23}^0 = S_2 \times L_2 = -4 \cdot 00 \times 12 \cdot 5 = -50 \text{ ft.-tons.}$$

$$M_{23} = -50 \times \frac{2 \cdot 65}{2 \times 2 \cdot 072} = -32 \cdot 0 \text{ ft.-tons.}$$

$$M_{32} = -50 \cdot 0 \times \frac{2 \cdot 30}{2 \times 2 \cdot 072} = -27 \cdot 8 \text{ ft.-tons.}$$

Third story from the top. Shearing force

$$S_3 = S_2 \times m_3 - (P_3 \sin \alpha + N_3 \cos \alpha) \times m_3.$$

$$S_3 = -4 \cdot 00 \times 0 \cdot 74 - 2 \cdot 0 \times 0 \cdot 96 \times 0 \cdot 74 = -2 \cdot 96 - 1 \cdot 42 = -4 \cdot 38 \text{ tons.}$$

$$M_{34}^0 = -4 \cdot 38 \times 12 \cdot 5 = -54 \cdot 8 \text{ ft.-tons.}$$

$$M_{34} = -54 \cdot 8 \times \frac{2 \cdot 74}{2 \times 2 \cdot 288} = -32 \cdot 8 \text{ ft.-tons.}$$

$$M_{43} = -54 \cdot 8 \times \frac{2 \cdot 48}{2 \times 2 \cdot 288} = -29 \cdot 7 \text{ ft.-tons.}$$

TABLE II.

MOMENT DISTRIBUTION TABLE									
JOINT	A_4	A_3			A_2			A_1	
MEMBER	A_4A_3	A_3A_4	A_3B_3	A_3A_2	A_2A_3	A_2B_2	A_2A_1	A_1A_2	A_1B_1
STIFFNESS	–	0.0575	0.30	0.1158	0.0480	0.46	0.1434	0.03	1.00
DISTR. FACTOR	–	0.123	0.631	0.246	0.075	0.706	0.219	0.03	0.97
CARRY-OVER FACT		–1.35	–	–0.65	–1.54	–	–0.46	–2.17	–
FIXED-END MOM.	–29.87	–32.78	–	–27.75	–31.97	–	–30.32	–38.82	–
1ST. DIST	–	+7.45	+38.18	+14.90	+4.67	+43.98	+13.64	+1.18	+37.66
C.O	–10.08	–	–	–7.18	–9.69	–	–2.51	–6.31	–
2ND DIST.	–	+0.88	+4.53	+1.77	+0.92	+8.61	+2.67	+0.19	+6.12
C.O	–1.19	–	–	–1.42	–1.15	–	–0.41	–1.23	–
3RD DIST	–	+0.17	+0.90	+0.35	+0.12	+1.10	+0.34	+0.04	+1.19
C.O	–0.23	–	–	–0.18	–0.23	–	–0.08	–0.16	–
4TH DIST.	–	+0.02	+0.12	+0.04	+0.02	+0.22	+0.07	–	+0.16
TOTAL	–41.15	–24.26	+43.73	–19.47	–37.31	+53.91	–16.60	–45.13	+45.13
SHEAR TABLE									
L	12.5	12.5	20.0	12.5	12.5	13.0	12.5	12.5	6.0
M/L	–3.29	–1.94	+2.19	–1.56	–2.99	+4.14	–1.33	–3.61	+7.52
SHEAR	+5.23	–5.23	–4.37	+4.55	–4.55	–8.30	+4.94	–4.94	–15.04

TABLE III.

FORCES NORMAL TO COLUMN					FORCES PARALLEL TO COLUMN				
	A_4	A_3	A_2	A_1		A_4	A_3	A_2	A_1
Ex. V. + N cos α	–	–	–	–1.40	Ex. V. + N sin α	–	–	–	–4.80
Ex. H. + P sin α	–	+1.92	+1.92	+10.56	Ex. H – P cos α	–	–0.56	–0.56	–3.08
Upper col.	+5.23	+4.55	+4.94	–	Upper col.	–35.61	–30.85	–22.32	–
Beam + S_b cos α	–	–1.22	–2.32	–4.21	Beam + S_b sin α	–	–4.80	–7.97	–14.44
Lower col.	–	–5.23	–4.55	–4.94	Lower col.	–	+35.61	+30.85	+22.32
Total	+5.23	+0.02	+0.01	+0.01	Total	–35.61	0	0	0

The moment distribution is shown in *Table* II, which also gives the shearing forces. The balance of the joints is checked in *Table* III.

The loads on the foundation are,

Horizontal:

$$H_A = +\,35{\cdot}61 \times 0{\cdot}28 + 5{\cdot}23 \times 0{\cdot}96 = +\,9{\cdot}97 + 5{\cdot}02 = +\,14{\cdot}99 \text{ tons.}$$

Vertical:

$$V_A = -\,35{\cdot}61 \times 0{\cdot}96 + 5{\cdot}23 \times 0{\cdot}28 = -\,34{\cdot}19 + 1{\cdot}46 = -\,32{\cdot}73 \text{ tons.}$$

Bridge Engineering in Britain.

The Minister of Transport stated in the House of Commons on February 3, 1960: "My visit to the continent satisfied me that British design (of bridges) and construction match work being done elsewhere. It may not therefore be necessary to make radical changes in present arrangements but, to ensure that full advantage is taken of the latest ideas and techniques, I am continuing to study whether it is desirable to try out new arrangements experimentally. I intend in any case that the partnership between my Ministry, bridge designers, and bridge builders should be made as close as possible."

Book Reviews.

"Dock and Harbour Engineering." Vol. 2. The Design of Harbours. By H. F. Cornick. (London: Charles Griffin & Co., Ltd. 1959. Price £6 6s.)

THIS book is the second volume of a work comprising four volumes which will replace the well-known books entitled "Dock Engineering" and "Harbour Engineering" by the late Brysson Cunningham. The new work retains what is of permanent value in the original books, which were first published more than fifty years ago, and includes much new matter in the collation of which the author has drawn on his own experience of harbour works, and has supplemented this by examples taken from the literature of the subject; knowledge of what has been done successfully or otherwise is of more value than theory when dealing with the forces of nature.

A brief history of harbours is followed by a description of the principles and factors affecting the design of harbours, breakwaters, pier-heads, jetties, and landing stages. Hydrographic surveying and the formation and maintenance of channels are included. There is a useful section on tides and waves, and much information is given on the forces exerted by waves and the distribution on walls of the pressures due to this cause. The section on piling is based on formulæ and data published elsewhere. A useful feature is the consideration of the impact of vessels on piled jetties, and details are given of many types of fenders. The construction of harbour works is intended to be the subject of a subsequent volume.

"Principles of Pavement Design." By E. J. Yoder. (London: Chapman & Hall, Ltd. 1959. Price 106s.)

THIS book of over five hundred pages, the text of which is based on the author's lectures, deals primarily with methods of the design of flexible and rigid roads and airfield runways used in the U.S.A. Construction is excluded except in so far as the procedure affects design. There appears to be a leaning towards computative design, based on more or less exact appraisal of the properties of the soil, magnitude of the loading, quality of the materials forming the road, and similar data, whereas elsewhere the tendency appears to be for experience to supplant calculations. A knowledge of soils mechanics and the making of concrete is assumed.

The question of whether or not to provide expansion joints in concrete roads is dealt with by tabulating the practice in the various states, from which it appears that in the majority of cases joints are no longer provided except in special positions, such as at bridges. If joints are provided in roads, the spacing is within the wide limits of 61 ft. and 1200 ft. Transverse contraction joints are commonly provided and the spacing is generally from 15 ft. to 40 ft., although in some cases the spacing is 100 ft. The effect of climatic conditions is noticeable in so far that the factors taken into account in design have different degrees of importance in districts where the weather is continually warm compared with localities where cold winters alternate with hot summers. There is a good chapter on defects which is comprehensive in its dealing with troubles encountered with flexible roads. There does not appear to be any direct comparison of rigid concrete and flexible roads as regards their relative first costs and maintenance charges.

"Berechnungsgrundlagen für Bauten." Compiled by E. L. Wedler. 23rd edition. (Berlin: Wilhelm Ernst & Sohn. 1959. Price 17 D.M.)

IN this edition German codes of practice relating to the design of buildings are given in full, with explanatory notes. Codes are now included for foundations, materials, rendering, timber construction, steel, combined concrete slab and steel girder construction, light-alloy structures, vibration of foundations carrying heavy machinery, and foundations in regions subject to earthquakes. It is proposed to give codes for concrete construction in a later volume.

"Planning." By S. Rowland Pierce, Patrick Cutbush, and Anthony Williams. (London: Iliffe & Sons, Ltd. Price 63s.)

COMPRISES 530 pages of notes on the planning of buildings for nearly all purposes, with more than 600 drawings.

"Staircase Manual." By J. A. Corns. (London: Cleaver-Hume Press, Ltd. Price 30s.)

A BOOK for carpenters with working drawings and descriptive notes on the construction of plain and ornamental, straight and curved, wooden staircases and handrails.

Exposed-aggregate Slabs in the South of England.

Mr. T. B. Smith, A.M.I.C.E., has contributed the descriptions in the following of some recent examples of precast slabs with exposed-aggregate surfaces which have been used on the façades of buildings in the south of England.

Some types of surfaces obtainable with exposed aggregate are shown in *Fig.* 1. The aggregates vary in size from fine sand to 6-in. flints. In selecting the most suitable size of aggregate it is important to consider the total area of the wall and the distance from which it will be viewed. The greater the distance from which the wall is normally viewed the larger should be the aggregate if the texture is to be effective. A contrast between the colours of the aggregate and the background, or the use of aggregate of two or more colours, is advisable if the wall will be seen at a distance. The aspect of the building and the shape and degree of exposure of the aggregate should be considered.

Fig. 2 is an example of the use of large aggregate, which in this case are 3-in. to 4-in. grey flints which have been rounded by use in the washmills of a cement works. The stones are set in a background of concrete with smaller aggregate of slightly lighter colour; the slabs have a plain margin of ordinary Portland-cement mortar. The slabs were cast face downwards on a bed of sand.

Broken flints (*Fig.* 3) set in a matrix of white cement mortar were used for the

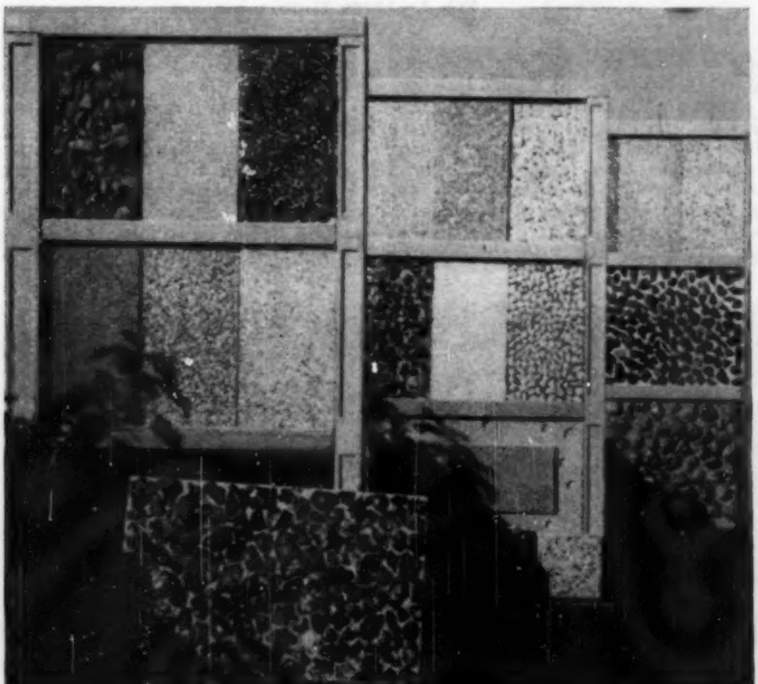


Fig. 1.—Some Types of Exposed-aggregate Surfaces.

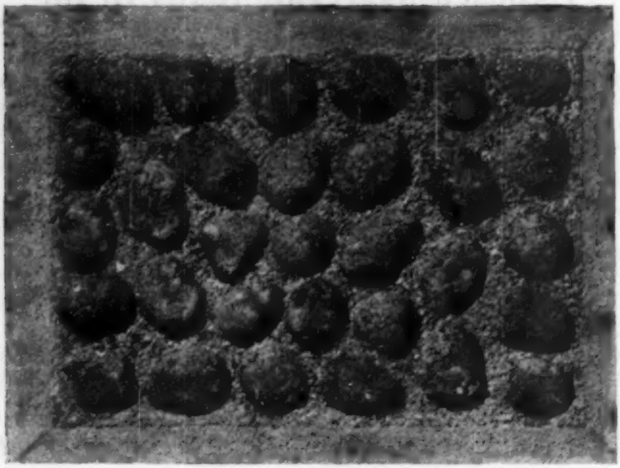


Fig. 2.—Large Flints.

slabs of a wall of a block of residential flats at Dover, for which Messrs. Dalgliesh & Pullen are the architects. In this case the slabs were cast face upwards and the stones were laid by hand on the backing concrete and then tamped to a level surface. Owing to the lack of experienced flint knappers the stones were broken by hand by unskilled men. The slabs in *Fig.* 4 were also cast face upwards, the large aggregate in this case being pieces of grey-gold Cornish slate ½ in. thick. This type of finish is also used for paving slabs.

When deciding the colour for exposed-aggregate slabs it is advisable to select a darker hue than that required for the finished work; large slabs in a wall generally appear lighter in colour than a small sample viewed at close quarters. The

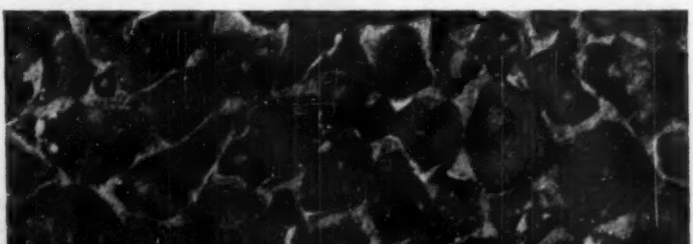


Fig. 3.—Split Flints set in White Cement Mortar.

colour of the backing concrete should be the same as that of the facing mixture, and the texture should be uniform. When using ordinary grey Portland cement care is needed to ensure that the colour is consistent. It may be necessary to obtain at one time sufficient cement to make all the slabs for one wall, or to blend the cement to obtain an even colour. Precautions must also be taken with the aggregates. Although some quarries guarantee the gradings of their materials, it may be necessary to screen the material before it is used. The colour of the stones from different faces of a quarry may differ, and before commencing to make a large number of exposed-aggregate slabs sufficient aggregate for the entire batch should be obtained, and it may be necessary to blend the materials.

Makers of exposed-aggregate panels generally have on display standard samples from which a customer can select one suitable for his purpose. It is generally possible to make other specimens with other textures and colours if none of the standard slabs is acceptable. It is important that a sample having an area of at least 4 sq. ft. should be examined before a decision is made. It is also necessary to realise that the surfaces of slabs made in large numbers are not likely to be as uniform as that of one slab made for the purpose of a specimen. Slight variations may, however, be desirable from the point of view of appearance. An example of a wall where the appearance is improved by a slight variation in colour is Strand School, Tulse Hill, London (*Fig.* 6), designed by the Architect's Department of the London County Council. In this example the natural appearance of green slate is simulated by using green aggregate in very small pieces.

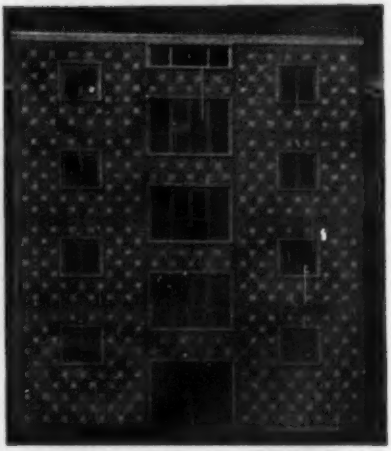


Fig. 5.—An Elevation of Dartford College.

The combination of exposed-aggregate and moulded patterns is shown in *Fig.* 5; in this case blocks of cast stone 6 in.

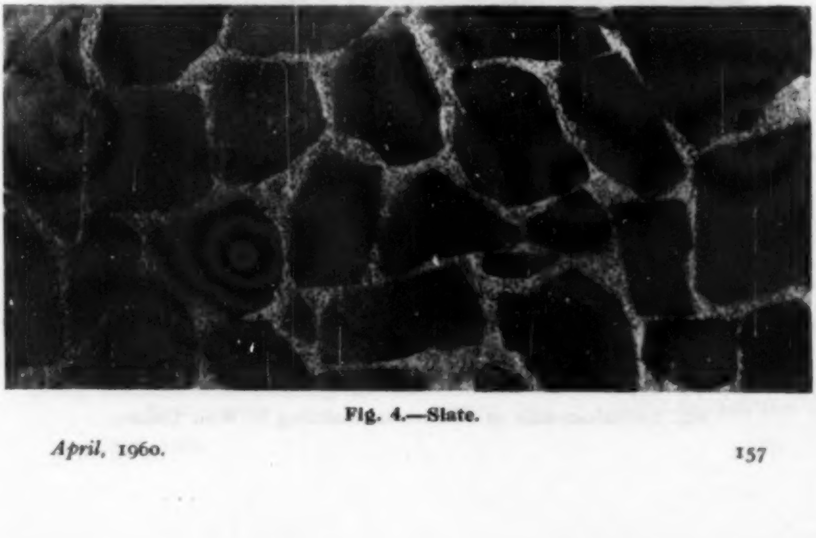


Fig. 4.—Slate.

Fig. 6.—Lift Enclosure at a School.

Fig. 7.—Balustrades at Residential Building in West London.

square are combined with exposed flint aggregate. This illustration is of Dartford Technical College, which was designed by Mr. S. H. Loweth, then County Architect for Kent. Similar panels are used on shops at the new town of Hemel Hempstead.

Slabs of completely contrasting colours and texture are frequently used for decorative effect by using different aggregates. In the example in *Fig.* 9, which illustrates the end wall of an office building in East London (Messrs. L. K. Watson & H. J. Coates, architects), the colours of the aggregates are light green and very dark blue.

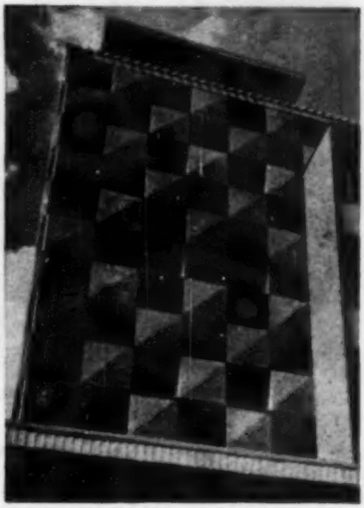


Fig. 9.—Office Building in London.

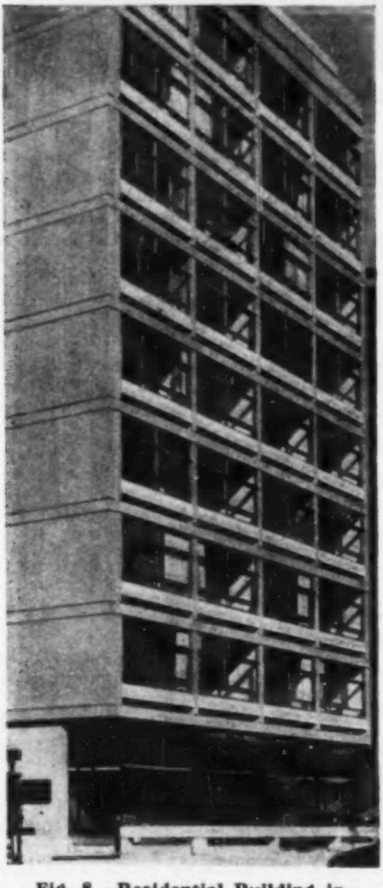


Fig. 8.—Residential Building in West London.

An example of covering the entire structural frame and walls of a building with exposed-aggregate slabs is shown in *Fig.* 8, which illustrates the new Hall of Residence for London University, the architects for which are Messrs. Richard Sheppard, Robson & Partners. On the side walls story-height slabs are provided between the beams at each floor-level. The beams in the side walls and the beams and columns on the front elevation are covered with precast slabs, as are also, in slabs of contrasting colour, the balcony beams. The surface of the slabs has the appearance of finely-axed light-grey granite. The upper part of the balcony beams comprises translucent quartz and white-cement concrete. The joints between the precast slabs are shown in *Fig.* 10, and *Fig.* 7 illustrates the balustrade at ground-level which has the same light-grey exposed granite finish as the members at higher levels.

The shape of the aggregate affects the tone of the slabs after weathering. Rounded aggregates are largely self-cleaning. Angular aggregates of rough texture tend to collect dirt, but this is

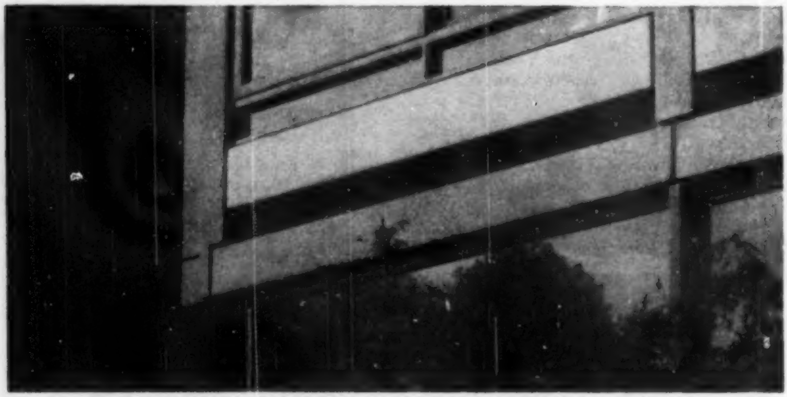


Fig. 10.—Detail of Residential Building in West London.

generally confined to the cement background; for this reason the area of exposed cement between the pieces of stone should be as small as practicable. It may be advisable for the cement background to be darker than the aggregate if the slabs are to be subjected to considerable atmospheric pollution.

Attempts have been made in Great Britain to improve the appearance and weathering qualities of exposed-aggregate slabs by the application of clear glazes, lacquers, and silicone washes. Some of these treatments tend to decrease the contrast in colour of the aggregates and backing, but it is not yet known how long these treatments are likely to be effective.

The aggregate, known as " Mineralite ", used for the slabs in *Figs*. 6 and 9, is a finely crushed aggregate generally of varied and brilliant colours. It is of Scandinavian origin and has been used for about thirty years on the Continent and for about seven years in Britain. It is supplied as a dry mixture of aggregate and cement. The final surface treatment in making slabs with this aggregate is to pat the face with a felt pad. It is claimed that the colours of the granites and other materials composing this aggregate cannot fade and that the surface does not craze.

The slabs described in the foregoing were made by Kendell's Stone & Paving Co., Ltd., with the exception of those in *Fig*. 5 which were made by the Atlas Stone Co., Ltd.

Research in Portugal.

THE following papers have been issued by the Portuguese Laboratório Nacional de Engenharia Civil, Lisbon. No prices are stated. The first three are printed in the English language and the others are printed in the Portuguese language with short summaries in English.

" The Design of Reinforced Concrete Beams ", by J. Ferry Borges and J. Arga E. Lima.

" Rectangular Staircases without Beams ", by J. Ferry Borges.

" Portuguese Experience on the Compaction Control of Earth Dams ", by Manuel Rocha, Armando Palma Carlos, José Folque, Virgilio Penalva Esteves.

" A Determinação de Coefficiente de Poisson do Betão das Nossas Barragens a Partir das Extensões Medidas In Situ ", by M. Q. Guerreiro.

" O Panorama Hidroeléctrico no Quadro do Desenvolvimento Industrial Brasileiro ", by J. Laginha Serafim.

" O Comportamento Térmico das Barragens de Betão ", by António Ferreira da Silveira.

Precast Construction of a Factory.

THE main building at a new factory for Messrs. Meggeson & Co., Ltd., at Camberwell, London, is about 195 ft. long and 165 ft. wide. There is also a canteen building of about 75 ft. by 30 ft. The structures are partly of two stories (*Fig.* 4). The first floor is designed for an imposed load of 200 lb. per square foot. To save time of construction as much of the work as possible was precast.

The primary beams are splayed in plan

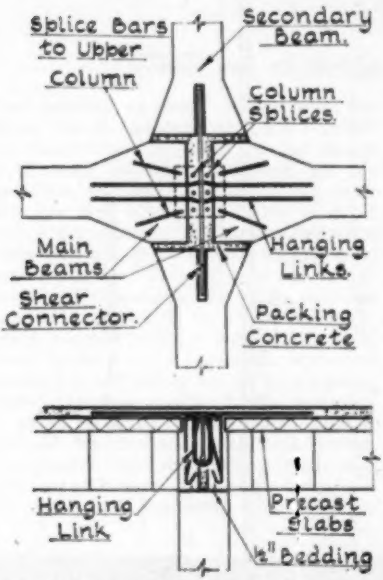


Fig. 1.—Connection of Internal Columns and Beams.

at the supports in order to provide sufficient resistance to compression without having to provide vertical haunches, which would decrease the clearance below the beams, and without having to provide reinforcement in compression. The columns are at 20 ft. centres in both directions and secondary beams are provided so that the floor slabs span about 10 ft.

The floor slabs are 6 in. thick and comprise 3 in. of concrete on 3-in. precast slabs reinforced with expanded metal. The upper slabs were cast in alternate

Fig. 2.—Precast Beams: showing Slots for Connections.

panels to reduce the effects of shrinking. Since the frame is composed entirely of precast members, little shrinking of the structure as a whole is expected.

The external columns were precast in one piece for the full height of the two stories. The internal columns are also precast and extend to the first floor only. Connecting the columns and secondary beams is a steel plate embedded in the column and inserted in a slot in the beam (*Fig.* 1). Inverted U-bars projecting from the top of the precast beams above the slots (*Fig.* 2) bear on the top edge of the plate. Deformed bars are welded to the top and bottom of the plates, through which holes are drilled to provide a bond when the joint is concreted. The slabs

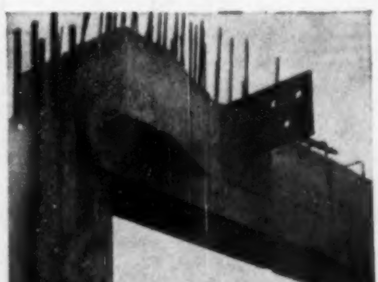


Fig. 3.—Main Beam on Column: showing Haunch and Connecting Plate.

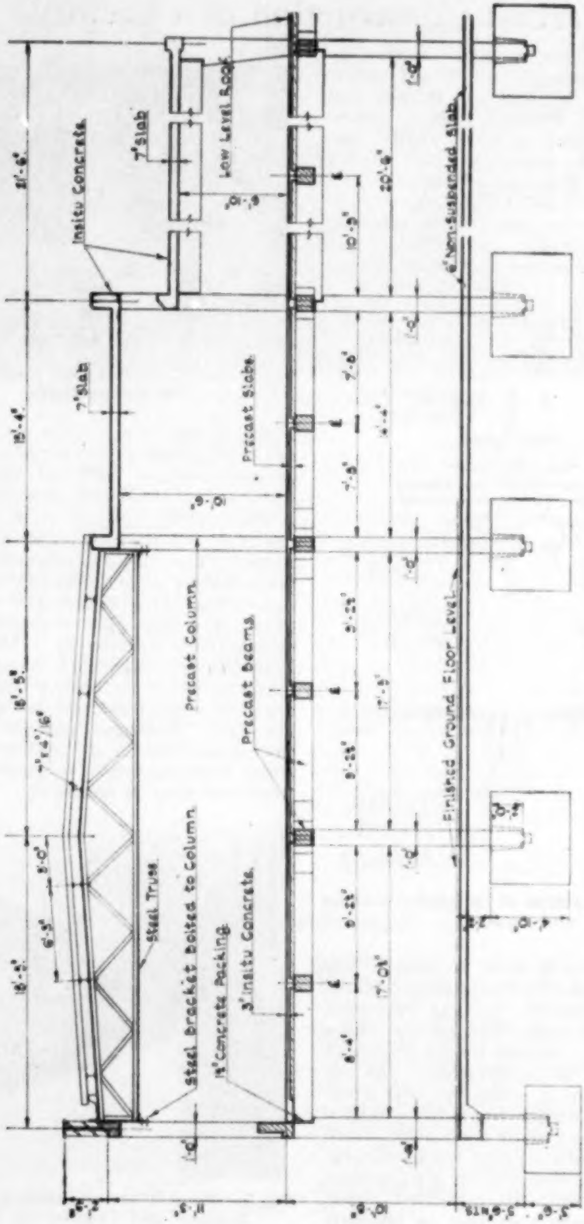


Fig. 4.—Typical Section.

adjacent to the external beams were cast in place, and are reinforced in the top and bottom to prevent the beams twisting.

The primary beams are supported on steps in the external columns (*Fig.* 3), and on the tops of the internal columns. Reinforcement projects from the external columns into the part of the primary beams which was cast in place with the floor slab. The connections between the primary and secondary beams are also by means of steel plates which are embedded in the primary beams (*Fig.* 3). The secondary beams are designed as being freely supported, but since liquid sugar used in the factory might be spilt on the floor, the secondary beams were made fully continuous to prevent cracking at the joints.

The architects are Messrs. Ronald Ward & Partners. The consulting engineers are Messrs. Hajnal & Myers, and the contractors were Messrs. H. Fairweather & Co., Ltd.

Widening of Bridge of Don, Aberdeen.

The existing masonry bridge over the River Don at Aberdeen was built between the years 1827 and 1830 to the design of Thomas Telford. Excluding the approaches, the structure is 520 ft. long and has five segmental arches each of 75 ft. clear opening. The arches, spandrel walls, and the slabs carrying the roadway are of granite. The width of the bridge between the parapets was 24 ft., and has been increased to 66 ft. The new work, which was completed in 1959, is separated from the old in order to prevent additional load being carried on the old foundations. Bored piles were used under the new piers to avoid vibration, which might damage the old foundations which had settled. The granite slabs under the original road and footpath were taken up and replaced by reinforced concrete slabs, thereby reducing the load on the old piers by about 1000 tons.

The new superstructure has reinforced concrete vault slabs, transverse walls, and deck slabs. The piers, voussoirs, and spandrel walls are faced with granite (*Fig.* 1), dressed to match the old bridge. The old parapets were taken down, the stones cleaned, and the parapets rebuilt, one in its original position and the other on the new structure.

The total cost, including roadworks, was about £282,000, of which £98,000 is the cost of the masonry. The engineer was Mr. A. G. Booth, City Engineer of Aberdeen, and the consulting engineers were Considère Constructions, Ltd. The main contractors for the work of reconstruction were Messrs. H. M. Murray & Co., Ltd.

Fig. 1.

Concrete at Bradwell Nuclear Power Station.

THE 300-megawatt nuclear power station being erected at Bradwell, Essex, for the Central Electricity Generating Board by the Nuclear Power Plant Co., Ltd., contains 200,000 cu. yd. of concrete which will be placed in the course of three years but most of which has been placed in a period of two years. There are many different types of concrete. The concrete used for structural purposes is classified as concrete of ordinary quality and special concrete of guaranteed density for shield-walls and the like. The most satisfactory mixtures using the aggregates available are determined in the contractors' main laboratory, but routine adjustments and testing are done at a laboratory on the site.

Ordinary Structural Concrete.

Particulars of the three principal mixtures of ordinary structural concrete are given in *Table* I and include the nominal volumetric proportions and the actual proportions by weight. The crushing strengths are the average strengths at 28 days of all cubes of each mixture. The standard deviations and coefficients of variation are given for each mixture. The aggregates are gravel and sand obtained from local pits. The slump is generally 2 in.

TABLE I.—ORDINARY STRUCTURAL CONCRETE.

Proportions		Maximum size of aggregate (in.)	Minimum specified crushing strength (lb. per sq. in.)	Crushing strength (lb. per sq. in.)	Standard deviation (lb. per sq. in.)	Coefficient of variation (per cent.)
Volumetric	By weight					
1 : 2 : 4	1 : 2·25 : 4·5	1½	3300	4500	440	9·8
1 : 2 : 4	1 : 2·15 : 3·6	¾	3300	4750	435	9·2
1 : 1½ : 3	1 : 1·8 : 3·2	¾	4125	5500	410	7·5

Concrete for Shield-Walls.

The concrete for walls of the reactors and elsewhere, where a shield against nuclear radiations is required, has a dry density of not less than 140 lb. per cubic foot and a compacting factor of 0·87. The aggregate is gravel brought by sea in barges from Brightlingsea because local aggregates are not sufficiently consistent in weight. The mixture is 1 : 2·5 : 6·1 by weight and the average crushing strength at 28 days is 4700 lb. per square inch with a standard deviation of 445 lb. per square inch and a coefficient of variation of 9·1 per cent.

Other concrete for the shielding construction has a minimum dry density of 165 lb. per cubic foot for which a mixed aggregate comprising gravel and barytes is used. The aggregate for concrete having a density of 200 lb. per cubic foot is entirely barytes. There is also a small quantity of special concrete having a minimum density of 340 lb. per cubic foot in which iron shot is used as aggregate. The densities of cubes of concrete for shielding were determined after drying to constant weight in an oven at 105 deg. C. The density of 9-in. diameter cores cut out of concrete in the work was the same (about 144 lb. per cubic foot) as that of the cubes.

The contractors for the civil engineering and building works are Sir Robert McAlpine & Sons, Ltd.

The Late Mr. Maxwell Ayrton.

THE death of Mr. Maxwell Ayrton, the architect, occurred in February at the age of 85 years. Mr. Ayrton was well known as an exponent of architectural concrete and contributed to this journal on that subject. The most notable of his earlier works, jointly with Mr. Simpson, was the British Empire Exhibition and Stadium at Wembley. Among other major reinforced concrete structures for which he was the architect are the bridge over the Thames at Twickenham and the Lea Valley viaduct. During the past decade he was in partnership with Mr. Courtenay Theobald.

Fig. 1.

A New Bridge over the Mersey near Manchester.

KINGSWAY bridge (*Fig.* 1) carries the Cheadle by-pass road over the River Mersey which, at the site of the bridge, is about 110 ft. wide. Provision is made for the sudden and considerable floods, to which the river is liable, by spans of 51 ft. 6 in. on each side of the main span which is 104 ft. long. The bridge, which is designed for Ministry of Transport standard loading and abnormal loading, carries two carriageways each 24 ft. wide, a central reservation 4 ft. wide, two 7-ft. footpaths and a verge 12 ft. wide adjoining each footpath, which enables the carriageways to be widened in the future to 36 ft. The overall width is 91 ft. 6 in. The angle of skew is 30 deg.

The bridge is a reinforced concrete structure, the two side spans of which cantilever about 24 ft. 6 in. over the river and support a suspended span of 55 ft. (*Fig.* 3). The piers are founded on good sandstone, which occurs at about 1 ft. below normal water level on the north side of the bridge and 6 ft. below normal water level on the south side. Excavation and construction of the foundations were carried out in cofferdams.

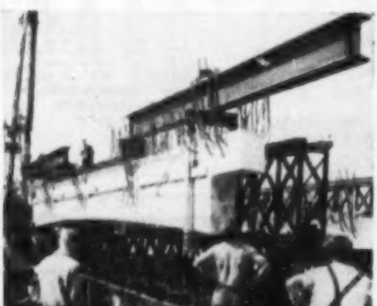


Fig. 2.—Erecting Precast Beams.

The girders, which are 2 ft. wide and vary in depth from 8 ft. at the piers to 5 ft. at the supports of the suspended span, were cast in place up to the level of the underside of the deck slab. Transverse beams are provided to ensure good distribution of the imposed load. The 10-in. deck slab was cast subsequently. There are no transverse beams under the central reservation and there is a gap 1 in. wide in the deck along the entire centre-line of the bridge.

The beams forming the suspended span are 4 ft. deep at the middle of their span, and were precast on an approach road and rolled on bogies along a temporary Bailey bridge erected between the cantilevers. The beams were lifted from the Bailey bridge by two mobile cranes (*Fig.* 2) and lowered into position. The Bailey bridge was moved for each pair of beams to reduce the distance the beams had to be lifted. The cantilevered girders are supported on the piers by mild-steel roller bearings in galvanised steel grease-boxes. The suspended girders are supported on rocker bearings at the north end and on roller bearings at the south end. A roller bearing at one end of the suspended span and a transverse joint through the deck and the road surface enable the deck to move due to changes of temperature. Mild-steel plates are embedded in the concrete on both sides of the joint which is covered by a phosphor-bronze plate

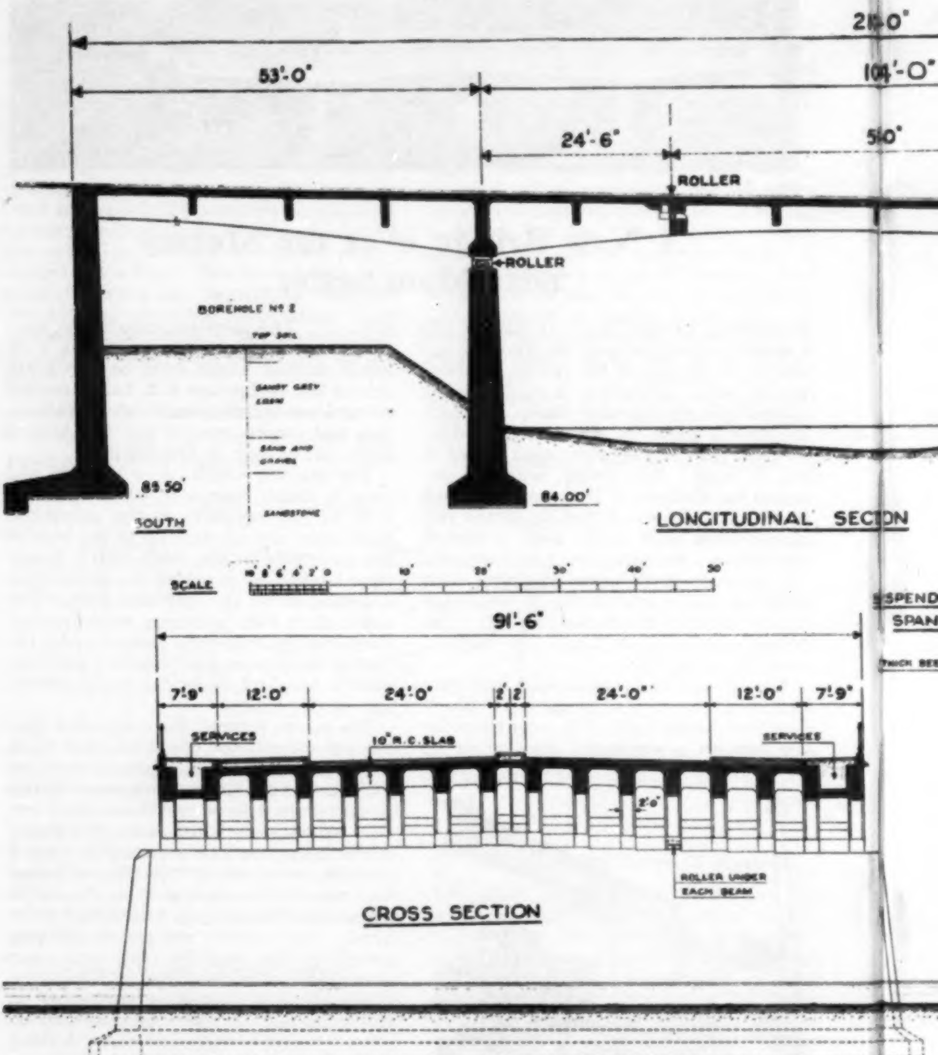
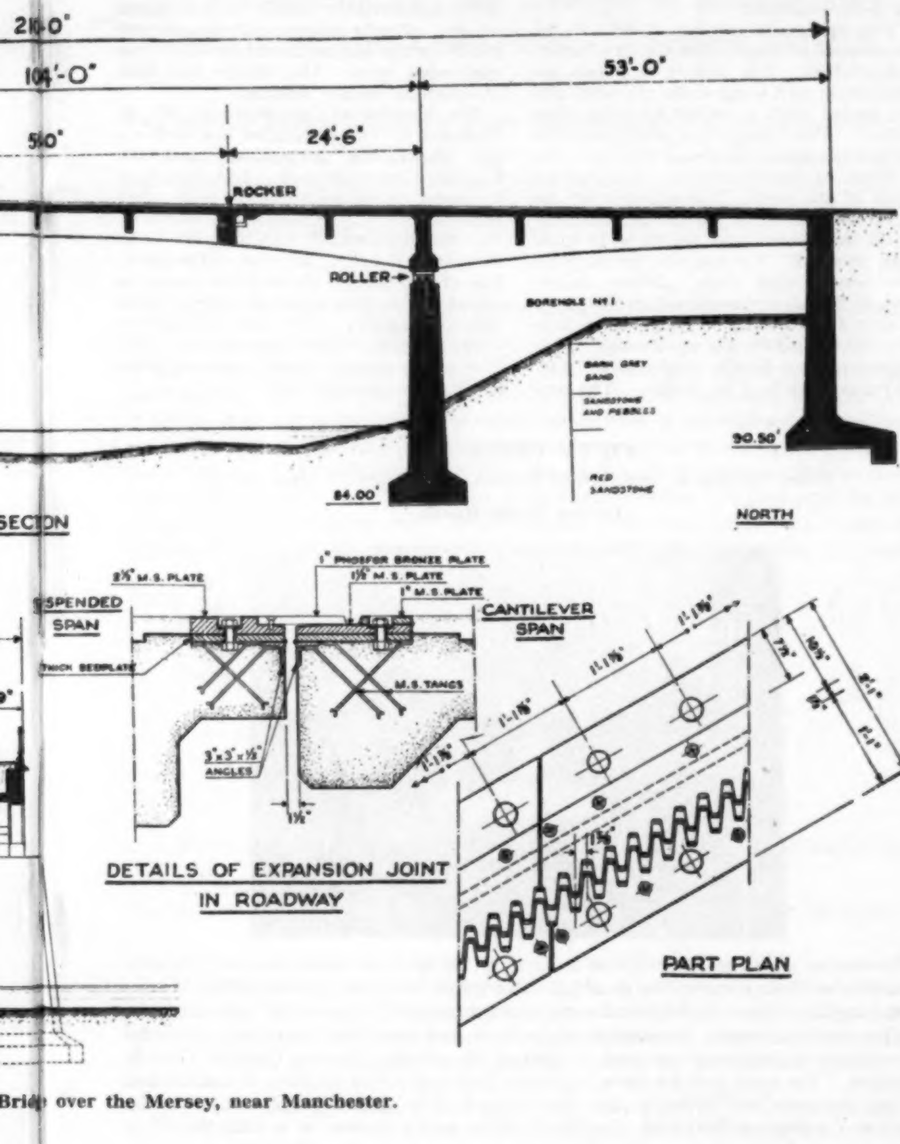


Fig. 3.—Section and Details of Brid[ge] over



Bri[dge] over the Mersey, near Manchester.

1 in. thick secured to the mild-steel plates by $\frac{3}{4}$-in. setscrews.

The wing walls are set in slots in the abutments, of which they are structurally independent. The spaces between the abutments and wing walls are filled and are sealed with a rubber-bitumen compound. The copings and pilasters of the wing walls are of Portland stone.

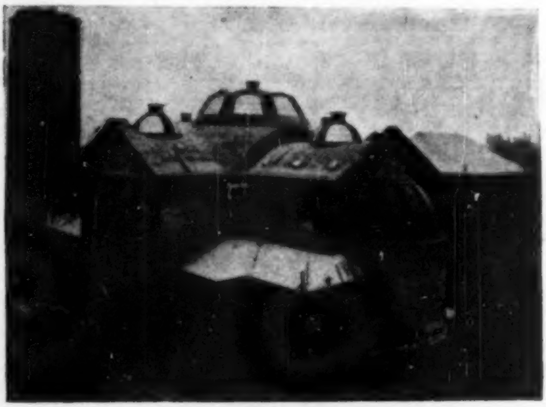
Wrought timber shutters were used for most of the work. The shutters for the faces of the exterior girders and the soffit of all the girders and beams were faced with plywood. The concrete for the piers, abutments, wing walls, girders, beams and deck had water-cement ratios of up to 0·55 and crushing strengths at 28 days of at least 3300 lb. per square inch. The aggregates are North Staffordshire sand and gravel up to $\frac{3}{4}$ in. in size. The concrete for the piers, abutments and wing walls and northern cantilever was mixed on site. Ready-mixed concrete was provided for the southern cantilever and the suspended span. The bridge was constructed in twenty months.

The Manchester City Surveyor, Mr. R. Nicholas, C.B.E., controlled the work for the Manchester Corporation and the Cheshire County Council. The consulting engineers for the bridge and its approaches were Messrs. L. G. Mouchel & Partners. The Manchester City Architect, Mr. L. C. Howitt, F.R.I.B.A., advised on the elevation of the bridge, the æsthetic design of which was approved by the Royal Fine Arts Commission. The main contractors were Tarmac, Civil Engineering, Ltd. The ready-mixed concrete was supplied by Trumix Concrete Ltd.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", April, 1910.

Barrel Vault Roofs.



"Reinforced concrete construction is gaining favour with architects for the construction of church roofs, the principal advantages being the practicability of constructing large barrel and domical roofs without unsightly trusses, tie rods, etc., and the low cost of upkeep. An example of this form, and one of the first to be constructed throughout of reinforced concrete, is that of St. Aloysius Roman Catholic Church, Glasgow. The roofs over the nave, transepts, and apse are in the form of semicircular arches, the nave roof having a clear span of 44 ft. 6 in., and a length of 65 ft. The nave roof is supported on arch ribs, reinforced in such a manner as to take the whole of the stress due to thrust without putting any thrust on the side walls, where the load is vertical; the ribs carry a $4\frac{1}{2}$-in. slab forming the roof covering."

Nomograms for the Design of Beams and Slabs by the Load-factor Method.—III.*

By J. C. STEEDMAN.

Rectangular Beams and Solid Slabs with Reinforcement in Tension only.

NOMOGRAMS NOS. 1C and 2C on pages 170 and 171 are supplementary to the nomograms given in this journal for February 1960, and apply to the design of rectangular beams and solid slabs respectively provided with reinforcement in tension only and if the permissible stress in the concrete is 1500 lb. per square inch.

Flanged Beams with Reinforcement in Tension and Compression.

Nomograms Nos. 6 and 7 (pages 172 and 173) aid the calculation of the area of the compression flanges of tee-, ell-, and I-beams and the area of reinforcement needed. The nomograms give results slightly more accurate than the Code, as the lever arm is measured to the centre of the area of the stressed concrete and not to the centre of the slab; it is therefore unnecessary for the designer to estimate the accuracy of the solution when the area of the rib is large in relation to the area of the flange as may be the case in hollow-tile slabs. If preferred the area of the compression flange may be determined from Nomogram No. 6 and the cross-sectional area of reinforcement required calculated from the expression $A_{st} = \dfrac{\text{bending moment}}{\left(d_1 - \dfrac{d_s}{2}\right)p_{st}}$ as in the Code.

The equations on which the nomograms are based are

$$\frac{M}{bd_1^2 p_{cb}} = \frac{2}{3}\left[\frac{d_s}{d_1}\left(1 - \frac{d_s}{2d_1}\right) + \frac{b_r}{b}\left(\frac{d_n - d_s}{d_1}\right)\left(1 - \frac{d_n}{2d_1} - \frac{d_s}{2d_1}\right)\right].$$

$$r_{st} = \frac{200 p_{cb}}{3 p_{st}}\left[\frac{d_s}{d_1} + \frac{b_r}{b}\left(\frac{d_n - d_s}{d_1}\right)\right].$$

When the value of $\dfrac{d_s}{d_1}$ is greater than the greatest value obtainable from the section $\left(\text{when } \dfrac{d_n}{d_s} \text{ is less than } 0{\cdot}5\right)$ reinforcement in compression must be provided and Nomogram No. 4A, 4B or 5 (see Part II) applies.

EXAMPLE No. 6.—An ell-beam having a rib 10 in. wide, an effective depth of 18 in., and a flange 30 in. wide and 4 in. thick is subjected to a bending moment of 1,600,000 in.-lb. Determine the cross-sectional area of reinforcement required if $p_{st} = 20{,}000$ lb. per square inch and $p_{cb} = 1000$ lb. per square inch.

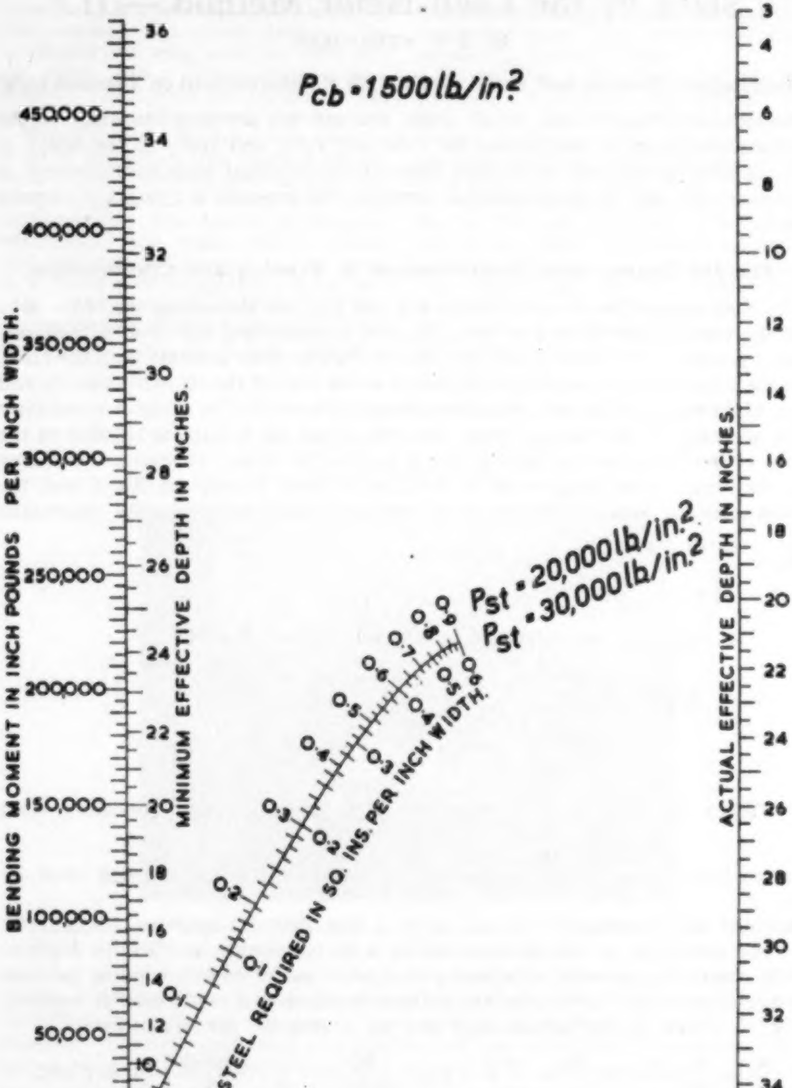
$$\frac{d_s}{d_1} = \frac{4}{18} = 0{\cdot}222;\quad \frac{b_r}{b} = \frac{10}{30} = 0{\cdot}333;\quad \frac{M}{bd_1^2 p_{cb}} = \frac{1{,}600{,}000}{30 \times 18^2 \times 1000} = 0{\cdot}165.$$

(*Continued on page* 174.)

* Concluded from March number.

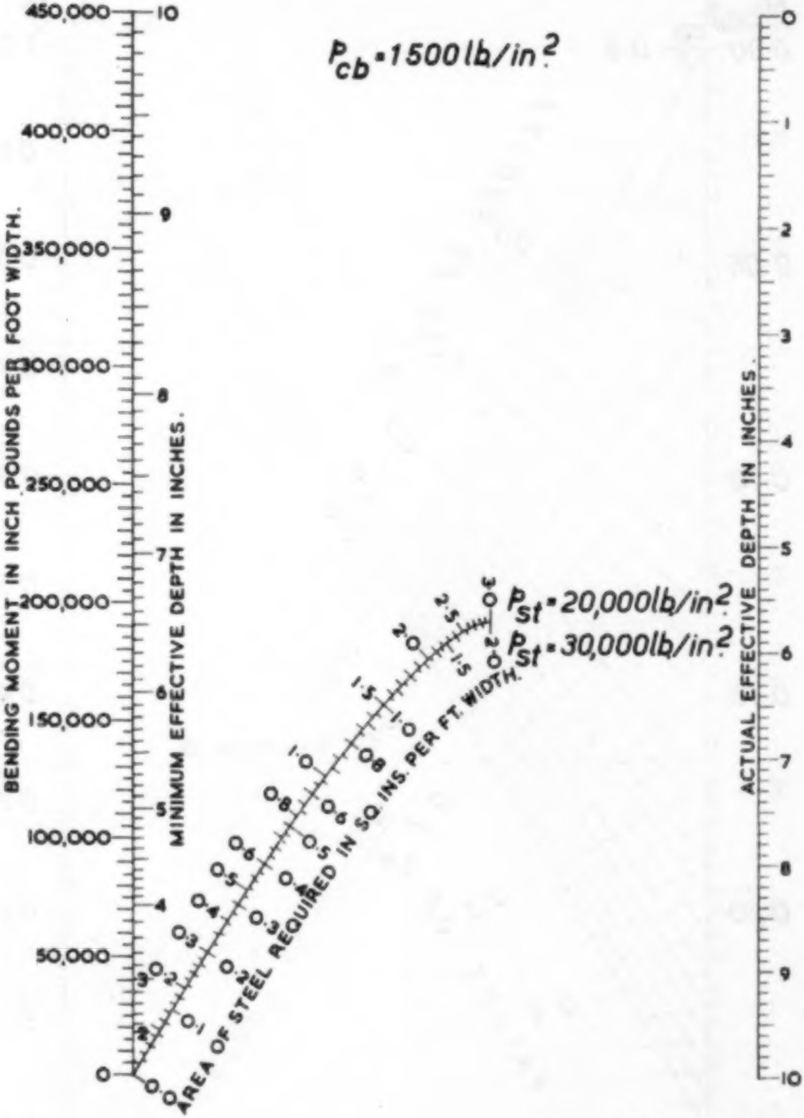
No. 1c.—DESIGN OF RECTANGULAR BEAMS BY THE LOAD-FACTOR METHOD.

Reinforcement in Tension Only.



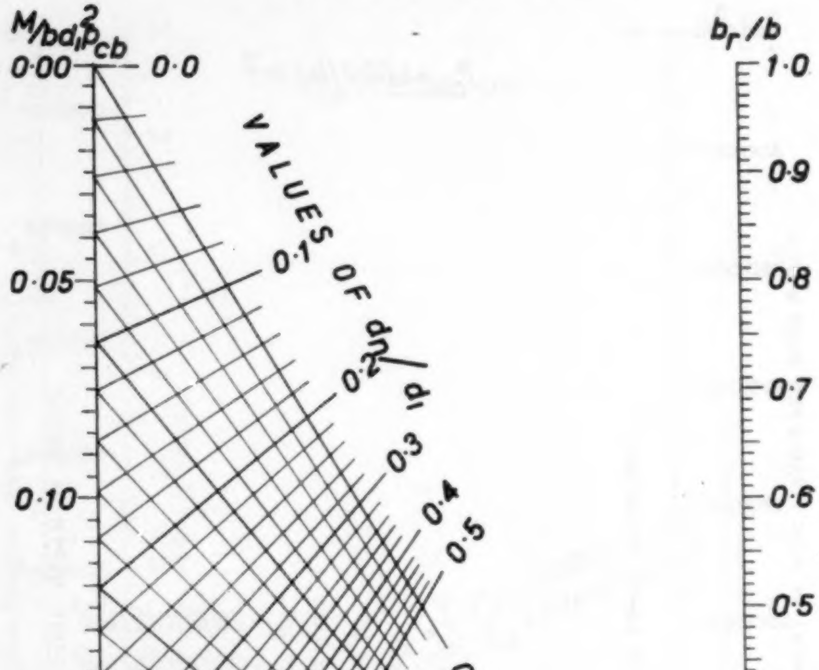
No. 2c.—DESIGN OF SOLID SLABS BY THE LOAD-FACTOR METHOD.

Reinforcement in Tension Only.



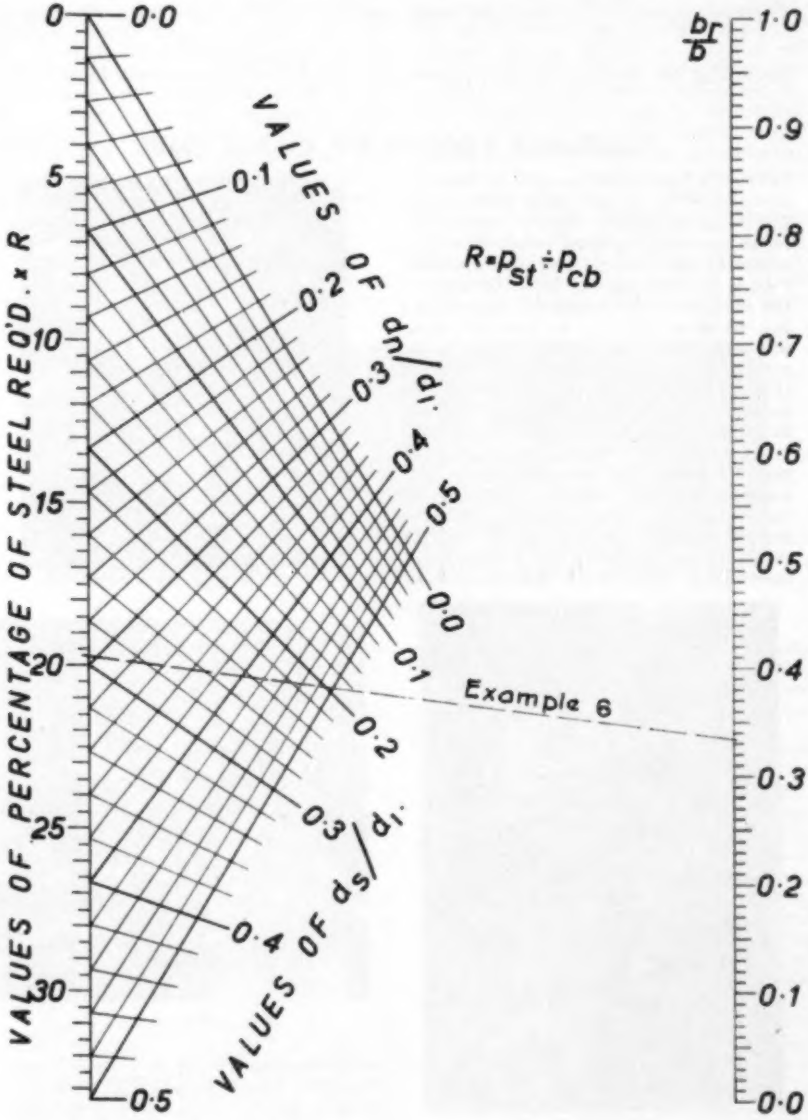
No. 6.—DESIGN OF FLANGED BEAMS BY THE LOAD-FACTOR METHOD.

Position of Neutral Plane.



No. 7.—DESIGN OF FLANGED BEAMS BY THE LOAD-FACTOR METHOD.

Reinforcement in Compression and Tension.



(*Continued from page* 169.)

From Nomogram No. 6, $\frac{d_n}{d_1} = 0{\cdot}450$ and, from Nomogram No. 7, $r_{st} \times R = 19{\cdot}75$.

Therefore $r_{st} = \frac{19{\cdot}75}{20} = 0{\cdot}99$ per cent. and $A_{st} = \frac{0{\cdot}99}{100} \times 30 \times 18 = 5{\cdot}33$ sq. in.

According to the formula in the Code, $A_{st} = \frac{1{,}600{,}000}{16 \times 20{,}000} = 5$ sq. in.

Cardboard Formers for Ribbed Slabs.

INVERTED cardboard boxes with internal stiffeners (*Fig.* 1) are being used in the U.S.A. as formers in ribbed slabs. The boxes are impregnated with wax and sealed at the factory with waterproofed tape 3 in. wide; at the construction site the bottoms of the boxes are dipped into hot paraffin (*Fig.* 1) to provide extra protection from the weather. The boxes are fixed to the shuttering by staples (*Fig.* 2) and the joints between them are covered with waterproofed tape; about five minutes are required to fix each box. It is claimed that when they are in position the boxes may be walked on in any weather without damage. When concrete has been placed and has hardened the shuttering is removed (*Fig.* 3) and the boxes can be pulled out by hand and used again. The cost in the U.S.A. of a box measuring 2 ft. by 2 ft. by 1 ft. 2 in. is 25 cents (about 1*s.* 9*d.*). The boxes are supplied by the Container Corporation of America.

Fig. 2.

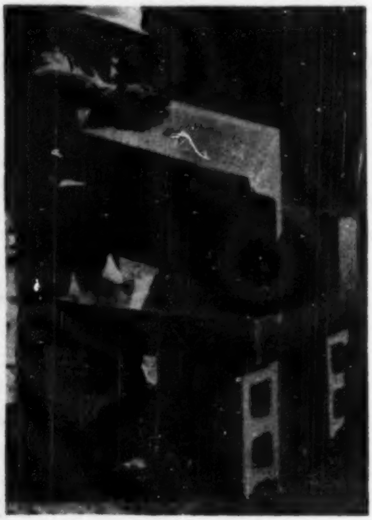


Fig. 1.

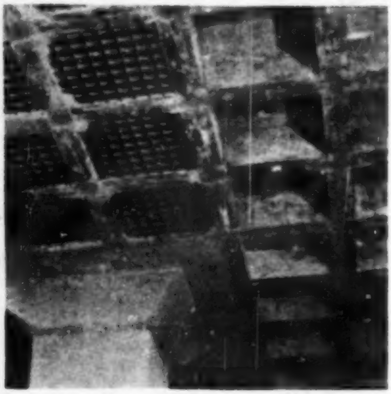


Fig. 3.

New Dams for Hydro-electric Works in Scotland.

SOME of the dams included in the hydro-electric works being installed by the North of Scotland Hydro-electric Board, which have been completed recently, or are in course of construction, or are about to be commenced are described in the following.* The works for the Conon Valley and Breadalbane schemes are nearing completion. Work has started on the Loch Awe scheme and the Strathfarrar and Kilmorack scheme.

Conon Valley Scheme.

Constructional work on the Conon Valley hydro-electric works in Ross-shire, which was commenced in 1947 and was completed recently, comprises several dams, twenty miles of tunnels, fifteen miles of aqueducts, six power stations, and extensive ancillary works. The main dams are at Fannich, Vaich, Glascarnoch, Meig, Luichart, and Torr Achilty and in Glen Orrin. There are smaller dams at Loch Droma and Loch a' Chuillinn, and a barrage at Achanalt. The generating station associated with the Fannich dam has been in operation since 1950.

The Vaich dam, which is an embankment of earth and rock with a core-wall of concrete, is 780 ft. long and has a maximum height of 80 ft. The Glascarnoch dam (*Fig.* 1) is of similar construction for part of its length of 1670 ft., but the central part 800 ft. long is a concrete gravity structure, founded on rock (Highland schist), and forms the spillway. The greatest height is 100 ft. and the width at the base of the central part is 90 ft. The bridge which carries a road over the spillway is of reinforced concrete and comprises twenty-eight spans each of 26 ft. The total quantity of concrete in the central part including the bridge is 185,000 cu. yd. The Meig dam, which is 540 ft. long and has a maximum height of 64 ft., is in part an earth embankment with a concrete core-wall, in part a gravity concrete structure, and elsewhere a concrete buttress dam. The Fannich and Loch Droma dams are rock-fill structures with a sealing membrane on the upstream faces.

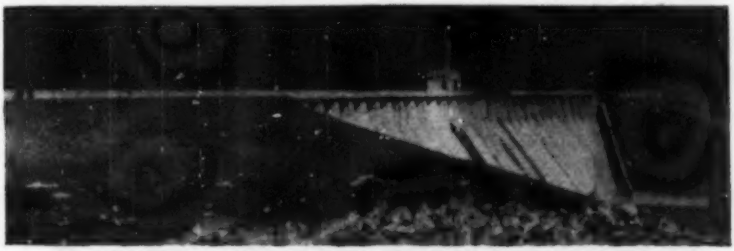


Fig. 1.—Glascarnoch Dam.

The Luichart dam, which raises the level of the water in Loch Luichart by 40 ft., is a gravity structure of plain concrete and is 60 ft. high and 720 ft. long; the width at the base is 40 ft. The quantity of concrete is 41,000 cu. yd. The bridge over the spillway, which comprises twenty-eight spans each 26 ft. long, extends the entire length of the dam and has a prestressed concrete deck, utilising the Freyssinet and Lee-McCall systems; this structure is one of the first prestressed concrete structures erected for the North of Scotland Hydro-electric Board. (The central section of the Allt-na-Lairige dam is also prestressed.* A fish-pass of the Borland type is provided.

The Torr Achilty dam incorporates the power-house and is similar to the

* Many of the dams in earlier schemes, which are now complete, are described in past numbers of this journal as follows. Loch Sloy dam: December 1948 and December 1950. Clunie and Pitlochry dams: January 1949, February 1949, and December 1950. Mullardoch and Benevean dams (Glen Affric scheme): February 1951. Cluanie and Loyne dams (Glen Moriston scheme): August 1954, January 1955, and December 1956. Allt-na-Lairige dam: July 1956. Loch Quoich dam: December 1956.

dam at Pitlochry on the Tummel-Garry works.* The width at the base is 40 ft. and the total quantity of concrete is 22,000 cu. yd.

The dam at Glen Orrin is 1080 ft. long and has a maximum height of 150 ft. It is a concrete gravity structure incorporating a Borland-type fish-pass.

The barrage at Achanalt is a reinforced concrete structure in which there are four gates each 30 ft. wide, a free spill-way 33 ft. long, and a weir semicircular in plan and 370 ft. long. The quantity of concrete is 1600 cu. yd. A fish-lock is provided between the gates.

The consulting civil engineers for the Conon Valley works are Sir Alexander Gibb & Partners. The contractors are as follows. Vaich, Glascarnoch, and Luichart dams and the Achanalt barrage: Messrs. Reed & Mallik, Ltd. Meig and Glen Orrin dams: Messrs. Duncan Logan, (Contractors), Ltd. Torr Achilty dam: Messrs. William Tawse, Ltd.

Breadalbane Scheme.

Construction of the Killin section of the Breadalbane hydro-electric works in West Perthshire is nearing completion and includes three dams.

Lubreoch dam, which (*Fig.* 4) raises the water-level in Loch Lyon by 77 ft. to form the main storage reservoir in this project, is a massive-buttress dam 1740 ft. long. The maximum height above the foundations is 130 ft. and the greatest width at the base is 80 ft. The quantity of concrete is 160,000 cu. yd.

Stronuich dam is a solid gravity dam 702 ft. long and has a maximum height of 40 ft. The greatest width at the base is 30 ft. The quantity of concrete is 21,000 cu. yd. A fish-pass is provided.

The Giorra dam (*Fig.* 2) will raise the level of Loch Giorra by 90 ft. It is a massive-buttress dam 1540 ft. long and has a maximum height of 110 ft. The width at the base is 80 ft. and the quantity of concrete is 91,000 cu. yd.

Lochan-na-Lairige dam in the Lawers section of this project, which was commenced in 1951 and was completed a few years ago, is also a massive-buttress dam 1100 ft. long and has a maximum height of 130 ft. The dam, which is flanked by an earth embankment 300 ft. long, has raised the level of the water in the loch by 90 ft. The length of the spillway weir is 54 ft. 6 in. and is designed to deal with 1280 cu. ft. per second.

Fig. 2.—Giorra Dam.

In the St. Fillans section of this scheme, the Lednock dam (*Fig.* 3), which was completed in 1957, is a diamond-head buttress dam 950 ft. long having a maximum height of 129 ft. The greatest width at the base is 120 ft. and the quantity of concrete is 110,000 cu. yd. The length of the spillway is 200 ft.

The consulting civil engineers for the Killin and Lawers works are Messrs. James Williamson & Partners, Ltd. The contractors for the Lubreoch dam are Messrs. James Miller & Partners, Ltd. The contractors for the Stronuich and Giorra dams are Messrs. Edmund Nuttall, Sons & Co., (London), Ltd. The contractors for the Lochan-na-Lairige dam were the Cementation Co. Ltd. The consulting engineers for the St. Fillans works were Sir M. MacDonald & Partners, and the contractors for the Lednock dam were Messrs. Taylor Woodrow Construction, Ltd.

* See footnote on page 175.

Fig. 3.—Lednock Dam.

Strathfarrar and Kilmorack Scheme.

The main civil engineering contracts for the hydro-electric works at Strathfarrar and Kilmorack in the counties of Inverness and Ross and Cromarty were awarded early in 1959. The main dam, which will be an arch dam 110 ft. high, will be about a mile downstream from Loch Monar in Strathfarrar.

There will be a subsidiary dam 50 ft. high near by. These dams will raise the level of the water in the loch by 75 ft. Water will then pass through a tunnel 5¾ miles long to a power station at the west end of Loch Beannachran. Near the outlet of the loch there will be a small dam by means of which the level of the water in the loch will be raised by 8 ft.

There will also be two dams each about 50 ft high on the River Beauly, one near the Crask of Aigas and the other at Kilmorack; these dams are combined with the power stations. The entire works including power stations, tunnels, dams, aqueducts, and ancillary works will take about five years to construct. The estimated cost is £14,250,000. Fish-passes operating on much the same principle as a canal lock will be installed at the Beannachran, Aigas, and Kilmorack dams.

The consulting civil engineers for the Strathfarrar and Kilmorack schemes are Sir William Halcrow & Partners. The Mitchell Construction Co., Ltd., are the contractors for the two dams at Loch Monar. Messrs. Duncan Logan (Contractors), Ltd., are the contractors for the Beannachran dam. Messrs. A. A. Stuart & Sons (Glasgow), Ltd., are the contractors for the Aigas and Kilmorack dams.

Loch Awe Scheme.

Some of the civil engineering contracts for the Loch Awe hydro-electric works were awarded in the autumn of 1959. Part of this scheme will be the first large pumped-storage plant to be constructed in

Fig. 4.—Lubreoch Dam.

Scotland. The scheme is in three parts. The works of the Inverawe section will regulate the level of the water in Loch Awe by a low barrage at a short distance downstream from the outlet of the loch, the level of which will not be raised. From the barrage, where there will be a fish-pass, the water is taken through a tunnel to a power station.

The Cruachan section includes the pumped-storage plant where electrical energy from the nuclear-power and steam-operated stations at periods of low load will be used to pump water from Loch Awe up to a high-level reservoir.

The Nant section includes the raising of the level of the water in Loch Nant by the construction of a dam near the outlet of the loch.

The consulting engineers for the Inverawe and Cruachan sections are Messrs. James Williamson & Partners, Glasgow, and for the Nant section, Messrs. Babtie, Shaw & Morton. The first contract for the Inverawe section has been placed with Messrs. George Wimpey & Co., Ltd.

A Prestressed Bow-string Pipe-bridge.

THE erection of a pipe-bridge (*Fig.* 1) with a span of 210 ft. over the River Tees at Eaglescliffe was completed in 1959 for the Tees Conservancy Board. It is a bow-string structure constructed of precast blocks which were prestressed after erection. The width of the deck, which carries two 33-in. water-mains and also provides a walkway, is 15 ft. The width of the arch boom is generally 7 ft., but bifurcates near each springing.

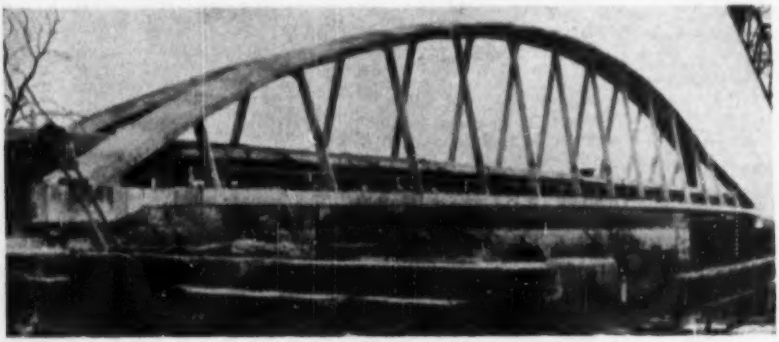


Fig. 1.

The structure was constructed and prestressed on the river bank, and was moved into its position across the river. A Bailey bridge was erected at one side of the site so that it extended over half the width of the river, and was supported on a temporary pier in the middle of the river. The prestressed concrete structure, mounted on bogies, was hauled by winches along the top of the Bailey bridge. When the leading bogie was over the central temporary pier the Bailey bridge was floated out on pontoons and into position over the other half of the river. The concrete bridge was hauled across the remaining width of the river, moved sideways into a position above the abutments which had been constructed in advance, and lowered by jacks on to roller bearings.

The total cost of the structure, including the piers and other foundations, was £54,000, of which £17,000 was for the superstructure and £2000 for the handrails and finishes. The contractors were the Dowsett Engineering Construction Co., Ltd., who collaborated in the design with the British Reinforced Concrete Engineering Co., Ltd. The prestressing was by the Gifford-Udall method, and the precast blocks were made by Dow-Mac Products, Ltd.

A Tunnel Lining with Dry Joints.

In *Figs.* 1 and 2 is illustrated a method of lining a tunnel with precast segments adopted for the water main from Hampton (Middlesex) to Chingford (Essex). The main, tunnelling of which was recently completed by the Metropolitan Water Board, is 19 miles long and has a diameter of 8 ft. 6 in., and is at such a depth that water will flow through by gravity. Several shafts of 12 ft. diameter are provided for the purposes of construction and inspection, and range in depth up to 192 ft. The shaft at the eastern terminal is 67 ft. 6 in. in depth and 25 ft. in diameter and accommodates the pumps which raise the water from the tunnel to the reservoirs. It will at first convey 70,000,000 gallons of water daily, but it is possible to increase the capacity up to 120,000,000 gallons daily.

The tunnel was constructed in sections each of which formed a separate contract, and is lined with segments which mainly have the shape and dimensions shown in *Fig.* 3, and are wedged together as shown in *Fig.* 2. The segments were made with sulphate-resistant cement.

The segments, which are called "Donsegs", were precast in a yard where more than 400,000 were made and were delivered to the working sites at a rate of 400 to 500 rings per week; each ring comprises twelve segments.

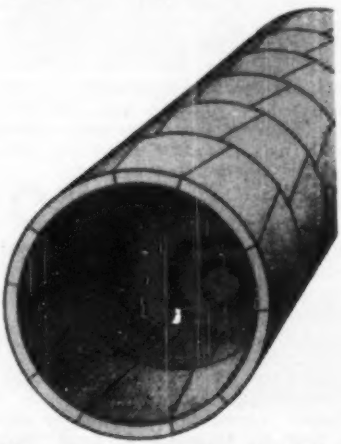


Fig. 1.—Concrete Segments forming Linings.

The method of construction, which it is claimed was used for the first time in this country for this tunnel, incorporates some novel features, including the lining which eliminates bolting and grouting, and an excavating shield which combines the operations of boring the tunnel and expanding the segments into position. The shield attained speeds in excess of 250 ft. per week, which greatly exceeds that obtained by the conventional method using bolted iron segments as used in London tube railways and is several times faster than the rate generally achieved on main drainage works.

The shield excavated the firm clay, in which the tunnel is bored, and formed a cavity of about 10 ft. diameter to suit the lining without having to place filling behind the lining. The segments, which are mainly interchangeable, were placed loosely by hand on temporary templates

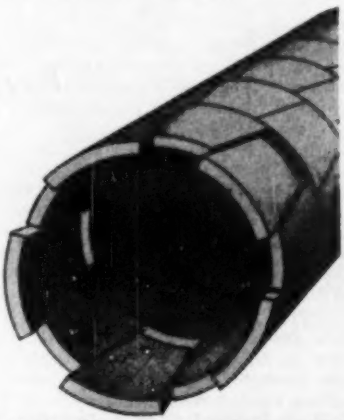


Fig. 2.—Segments before Ramming.

Fig. 3.—Precast Segment.

to form a ring comprising twelve segments, which were then wedged tightly together by means of twelve hydraulic rams acting from the shield. The surface of the lining was then sprayed with ½-in. cement mortar which was trowelled smooth by a mechanical process.

The work was carried out under the general direction of the Chief Engineer to the Board, Mr. W. M. Lloyd Roberts, and his predecessor, Mr. H. F. Cronin, C.B.E., M.C. The consulting engineers were Sir William Halcrow & Partners. The main contractors for the tunnel were Messrs. Kinnear, Moodie & Co., Ltd., who made many of the segments; the remainder were made by Stent Precast Concrete, Ltd. The contractors for the shafts and other works were Messrs. William Press & Son, Ltd., and Messrs. A. Waddington & Son, Ltd. The spraying of the lining was done by the Turiff Construction Corporation, Ltd.

[A description of an experimental tunnel 1000 ft. long which preceded the construction of the entire works was given in this journal for April, 1954, based on a paper by Mr. P. A. Scott in the Proceedings of the Institution of Civil Engineers, May, 1952. Each ring in this tunnel comprised ten segments (as shown in *Figs.* 1 and 2).

Another method of lining tunnels with concrete segments without bolting or grouting is to be used for an experimental railway "tube" tunnel to be driven by London Transport Executive in north-east London. The segments are erected to form a ring into a gap in which a key-segment is forced. The contractors for this trial work are Messrs. Kinnear, Moodie & Co., Ltd. A similar method, which was used for the new main-line railway tunnels at Potters Bar, Herts., is described in this journal for May, 1958.]

Concrete Roads in the U.S.A.

STATISTICS published annually by the U.S. Bureau of Public Roads show that the total length of concrete roads in the U.S.A. was about 95,000 miles in 1942 but only about 75,000 miles in 1957, although during this period about 19,000 miles of concrete roads were constructed. According to the latest figures available * the total length of surfaced roads in the forty-nine States exceeded 600,000 miles, so that the length of concrete roads was about one-eighth of the total at the end of the year 1957. About 2000 miles of concrete roads were constructed during 1957, including about 380 miles of new roads or earthen roads paved with concrete, about 1000 miles of old concrete roads resurfaced with concrete, and about 500 miles of existing roads having other surfaces that had been changed to concrete. Because about 4000 miles of concrete roads had been changed to other types of surfacing, mainly bituminous or asphaltic, there was a reduction during the year of about 2000 miles of roads with concrete surfaces.

The exact lengths of each type of road are not available because the lengths of roads paved with bricks or blocks are included in the lengths of Portland-cement concrete roads and the data on changes of types of road do not indicate whether the bituminous or asphaltic surfaces were applied to existing concrete roads or whether the concrete roads were replaced entirely by another form of construction. Also, information on roads having a lean concrete sub-base surfaced with bituminous or similar materials is separate.

In 1958 the cost of concrete roads was about 50 per cent. higher than in 1946 and had remained almost unchanged since the middle of 1956. Comparison with costs in previous years, and especially before 1940, is not possible because of the more stringent requirements in the later years. In the period from 1946 to 1958 the cost of excavation increased by about 15 per cent., whereas the cost of reinforced concrete structures such as bridges had increased 62 per cent. by 1957 but fell to about 53 per cent. in 1958.

* "Highway Statistics, 1957." Published in 1959 by the U.S. Department of Commerce.

Symposium on Grouting.

THE date of the Symposium on Grouting Prestressed Concrete, which is being organised by the Fédération Internationale de la Précontrainte and R.I.L.E.M., has now been changed to January 5 to 7, 1961. As previously announced, the Symposium will be held at Trondheim, Norway, and the programme is not altered. Contributions of papers should be sent to Professor I. Lyse, Norges Tekniske Hogskole, Trondheim, Norway, and should reach him before August 15.

Deterioration of Concrete due to Alkali Reaction.

OBSERVATIONS of the deterioration of concrete structures have led to the discovery during the last decade that this is sometimes caused by a chemical reaction between the alkalis present in cement and soluble silica which is present in certain aggregates. Most British aggregates are inert but some European and American aggregates are not. A committee appointed in Denmark in 1953 to study the problem has issued two interim reports * from which these notes are abstracted.

The alkali reaction occurs between cement with a high content of alkali and aggregates containing soluble silica, when alkaline hydroxide reacts with the silica and water to form an expansive gel. If sufficient water and alkali are present the gel is eventually transformed into a solution of alkaline silicate. The reaction produces a measurable expansion of the concrete, cracks and fractures, craters in the surface, exudations of resin-like gel, and efflorescent deposits.

EXPANSION.—Expansions of 0·5 to 0·7

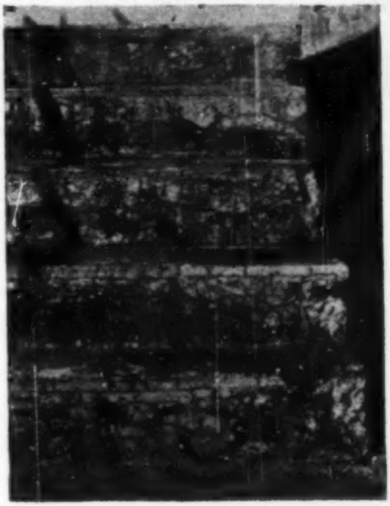


Fig. 1.

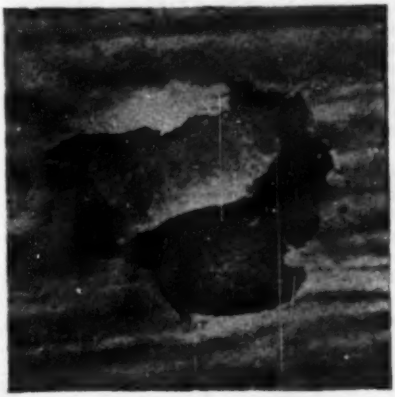


Fig. 2.

Fig. 3.

* Committee on Alkali Reactions in Concrete. Progress Reports N.1 (1956) "Disintegration of Field Concrete", by G. M. Idorn, and L.1 (1957) "Investigation of the Effect of Some Pozzolans on Alkali Reactions in Concrete", by A. H. M. Andreasen and K. E. Haulund Christensen. Copenhagen: The Danish National Institute of Building Research. Price 12 Kroner each. Printed in the English language.

per cent. are known to have occurred during periods of 14 to 15 years. The rate of expansion is greater in summer than in winter.

FRACTURES.—Cracks and fractures are invariably present. Crazing is frequently observed, and is often accompanied by spalling. Long wide cracks occasionally occur. The deterioration of the staircase shown in *Fig.* 1 took place in seven years, during which time the treads were re-surfaced with mortar.

CRATERS. — Particles of chemically-active aggregate which occur near the surface of the concrete may cause the formation of craters. *Fig.* 2 shows a crater which appeared in a jetty 25 years after it was built. The photograph was taken fourteen days after the surface was painted, when the crater was not present. Craters are always formed as a result of the development of gel.

EXUDATIONS OF GEL.—These occur frequently as drops up to 1 in. diameter on ceilings and walls, and as craters up to ¾ in. diameter on floors. Old deposits are usually white and dry. Fresh deposits are usually watery, viscous or hard, yellow or brown in colour, and resin-like in appearance. *Fig.* 3 shows a fresh drop 7 mm. in diameter, surrounded by older deposits.

SECONDARY DEPOSITS.—Deposits of calcium carbonate often occur on surfaces, forming stalactites under ceilings and beams (*Fig.* 4).

Some of these symptoms may also appear in concrete which has been damaged by frost or sea-water. *Table* I gives a comparison of the effects produced by each of these causes.

Fig. 4.

TABLE I.

CAUSE OF DETERIORATION	EVIDENCE OF DETERIORATION				
	EXPANSION	CRACKING	CRATERS	EXUDATIONS	DEPOSITS
ALKALI REACTION	FREQUENT	PATTERN CRACKS INVARIABLE SPALLING OCCASIONAL	FREQUENT	FREQUENT	FREQUENT
FROST	VERY RARE	SPALLING FREQUENT PATTERN CRACKS RARE	FREQUENT	NONE	FREQUENT
SEAWATER ATTACK	NONE KNOWN	SPALLING FREQUENT PATTERN CRACKS RARE.	NONE	NONE	FREQUENT

Accidents on Construction Work.

An accident that occurred recently during the course of construction of a steel bridge in the north-west of England brought attention to the fact that although there are strict safety regulations in the building industry there is none for civil engineering. The Minister of Labour announced in the House of Commons recently that it was proposed to extend the definition of "work of engineering construction" in the Factories Act to include most civil and constructional engineering.

In a "Report on Safety and Health in the Building and Civil Engineering Industries (1954 to 1958)" issued by the Minister of Labour in February, 1960 (and obtainable from Her Majesty's Stationery Office; price 1*s.* 6*d.*), it is stated that the number of accidents in 1958 reached the highest ever recorded. Two-thirds of the fatal accidents and nearly half of all accidents in building in that year, were due to falls of persons.

Details of some actual accidents are given in the number for January, 1960, of a quarterly publication entitled "Accidents: How they happen" (obtainable from Her Majesty's Stationery Office; price 1*s.* 3*d.*). This review deals with accidents in factories, on building sites and on works of engineering construction.

The Federation of Civil Engineering Contractors last year organised training courses for the promotion of safety. These are held in collaboration with the Ministry of Labour, in North London and Birmingham, but similar courses are being arranged to be held at Liverpool, in South Wales, and in Scotland. The syllabus is such that it is suitable for supervisors down to the level of ganger, and for young engineers. The subjects dealt with include the causes of accidents, tidiness on sites, excavations and earthworks, staging, electricity, lifting and carrying, and the cost of accidents. Dates of the courses can be obtained from the Federation at Romney House, Tufton Street, London, S.W.1.

Accidents in the Building Industry.

The following is a summary of a talk on "Preventing Falls in the Building Industry" given by Mr. J. A. Hayward at the National Industrial Safety Conference held at Scarborough last May.

Accidental falls in the building industry were causing some concern so long ago as 1858, in which year "The Builder" reported that the number of cases of falls treated at London hospitals in one year was not likely to be less than 15,000 and that a very large proportion of the sufferers were in the building trades. During recent years there have been an average of 15,000 accidents reported annually in the building industry and, of these, about 200 have been fatal. Although there was substantial improvement in the immediately preceding years, the number in 1958 was a little greater than the average of the past three or four years. About 86 per cent. of all fatal building accidents and about 50 per cent. of reportable injuries are the result of falls.

The casual nature of the building work makes it difficult to apply safety training schemes such as can be undertaken in many other industries. Men are subject to the vagaries of the weather. When a man has spent several hours on a building site, probably wet, cold, and tired, he may not be in a frame of mind to plan the safest way of doing a job, and this may be the reason why the accident rate is perhaps greater than in some other industries. If management and men take elementary precautions to see that safety regulations are complied with, many falls which now occur could be prevented. In one civil engineering and building firm where such precautions have been enforced less than 25 per cent. of the fatal accidents have been a result of falls, and of reportable accidents about 20 per cent. have been due to falls compared with the average of about 50 per cent. for the industry as a whole.

A Guide to Safety.

A booklet entitled "Organise for Safety" was published recently by the National Federation of Building Trades Employers and gives guidance to building firms on the formulation and application of a policy for the prevention of accidents.

The correct methods of erecting and maintaining scaffolding, operating cranes and lifting appliances, and guarding openings are well known, but the object of the

booklet is to show how the determination of the management to reduce the number of accidents may be translated into taking effective precautions by the development of a chain of responsibility the members of which include safety officers, agents, general foremen, trades foremen, gangers and other employees, each of whom must be clearly instructed as regards their personal responsibilities. The recommendations can be applied in principle by all building firms irrespective of their size since, while the complete safety organisation may not be appropriate to a small firm, the principles can be applied with such modifications as may be necessary to suit particular circumstances. Copies of the booklet are obtainable from the Federation, 82 New Cavendish Street, London, W.1 (price 2s. 6d. post free).

Disposal of Unwanted Mixed Concrete.

A SUPPLIER of ready-mixed concrete in Washington, U.S.A., has installed a machine in which the aggregate is recovered from mixed concrete that is left over at the end of the day. The machine is in the form of an inclined horizontal rotating drum in which is a screw conveyor. While it is in the transit-mixer water is added by the driver until the concrete has a slump of 10 in. or so. This material is tipped into the higher end of the drum in batches of ½ cu. yd. to ¾ cu. yd. The rotation of the drum stirs the material so that the cement and water and the finest sand are passed along the drum and out at the lower end. The screw-conveyor then transports the aggregate to the discharge end of the drum while it is sprayed with water to remove any remaining cement. After leaving the drum the aggregate is passed over screens and separated into coarse and fine material for re-use. It is stated that the aggregate so recovered is of the same grading as the original mixture except for the loss of some of the finest particles. The capacity of the machine is about 10 cu. yd. per hour. The water and cement are stored in sumps until the sludge has settled to the bottom, and the sludge is then scooped out and dumped into a worked-out part of the gravel pit.

Partnership.

MR. E. W. H. GIFFORD has taken Mr. J. R. Lowe and Mr. H. E. Lewis into partnership. The consulting engineering practice will henceforth be carried on under the name of E. W. H. Gifford and Partners.

Reinforced Lightweight Concrete.

RESEARCH on reinforced lightweight concrete at the Building Research Station was extended in the year 1958 to include aerated concrete as well as concrete made with foamed slag, expanded shale or clay, and sintered pulverised-fuel ash. Many specimens exposed in severely polluted atmospheric conditions for a year were broken open; corrosion due to exposure could not be observed on any of the bright mild-steel bars embedded in the specimens. Pull-out tests and tests on beams of lightweight concrete show that the bond strength was about one-half to three-quarters of the bond strength of gravel concrete having the same compressive strength. The bond of bars in a horizontal position during casting was only about one-half of the bond of vertical bars. The deflections, at working loads at mid-span of the lightweight concrete beams though somewhat greater than those of gravel concrete, were not considered to be excessive. The combined effect of shrinkage and creep on the deformation of shallow beams subjected to loading of long duration is such that the maximum deflections of lightweight concrete beams are greater than those of beams made with gravel concrete by between about 15 and 50 per cent.—" Building Research 1958." H.M. Stationery Office. Price 5s. 6d.

Brochure Received.

A NEW booklet entitled " Asbestos and Asbestos-Cement Products " deals with asbestos as a raw material, and asbestos-cement products, and gives notes on the properties and methods of using such products. The booklet is obtainable free from Turners Asbestos Cement Co., Ltd., Trafford Park, Manchester 17.

SITUATIONS VACANT. Consulting engineers require reinforced concrete detailers. Minimum three years' experience. Permanent positions. Pension scheme, luncheon vouchers. Send details of experience, and salary required, to F. R. Bullen & Partners Ltd., Dacre House, Dean Farrar Street, Westminster, S.W.1.

SITUATIONS VACANT. Junior assistants wanted for consulting engineers' office. Apply in writing to W. E. J. Budgen & Partners, 54 Queen Anne Street, London, W.1.

SITUATION VACANT. Reinforced concrete designer-detailer conversant with hollow tile floors, staircases, etc., required by specialist contractor in London. Write, giving brief particulars, and salary required, to Broadmead Products Ltd., 231 Strand, London, W.C.2.

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SITUATION VACANT. Senior engineer required for London professional office. Must have professional qualifications and at least five years' design experience in structural reinforced concrete. Knowledge of steelwork an advantage. Age limit 30–40. Candidates should be capable of handling contracts from start to finish, and able to control staff. The work is varied and offers scope for initiative. A good salary will be offered. Apply, stating experience, and salary required, to Farmer and Dark, Romney House, Tufton Street, London, S.W.1.

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SITUATIONS VACANT. Design engineers and draughtsmen required for extensive reinforced concrete heavy foundation work in S. Wales. Write, giving age, experience, etc., to Box 4661, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete draughtsmen required for contractor's office in Westminster. Applicants, age 25–35, must have had at least five years' experience in detailing with firm of repute. Written applications only, giving details of experience, to Tileman & Co. Ltd., Romney House, Tufton Street, London, S.W.1.

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